STEEL CONSTRUCTION

TUCKER

AMERICAN SCHOOL *of* CORRESPONDENCE

CHICAGO ILLINOIS

GUARANTY BUILDING (NOW CALLED PRUDENTIAL BUILDING), BUFFALO, N. Y.
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Steel Construction

A Practical Treatise on the

MODERN USE OF STEEL IN THE ERECTION OF FIREPROOF BUILDINGS,
AND ITS APPLICATIONS TO STRUCTURAL WORK
IN GENERAL

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ILLUSTRATED

CHICAGO
AMERICAN SCHOOL OF CORRESPONDENCE
1910

Foreword

IN recent years, such marvelous advances have been made in the engineering and scientific fields, and so rapid has been the evolution of mechanical and constructive processes and methods, that a distinct need has been created for a series of *practical working guides*, of convenient size and low cost, embodying the accumulated results of experience and the most approved modern practice along a great variety of lines. To fill this acknowledged need, is the special purpose of the series of handbooks to which this volume belongs.

¶ In the preparation of this series, it has been the aim of the publishers to lay special stress on the *practical* side of each subject, as distinguished from mere theoretical or academic discussion. Each volume is written by a well-known expert of acknowledged authority in his special line, and is based on a most careful study of practical needs and up-to-date methods as developed under the conditions of actual practice in the field, the shop, the mill, the power house, the drafting room, the engine room, etc.

¶ These volumes are especially adapted for purposes of self-instruction and home study. The utmost care has been used to bring the treatment of each subject within the range of the com-

mon understanding, so that the work will appeal not only to the technically trained expert, but also to the beginner and the self-taught practical man who wishes to keep abreast of modern progress. The language is simple and clear; heavy technical terms and the formulæ of the higher mathematics have been avoided, yet without sacrificing any of the requirements of practical instruction; the arrangement of matter is such as to carry the reader along by easy steps to complete mastery of each subject; frequent examples for practice are given, to enable the reader to test his knowledge and make it a permanent possession; and the illustrations are selected with the greatest care to supplement and make clear the references in the text.

¶ The method adopted in the preparation of these volumes is that which the American School of Correspondence has developed and employed so successfully for many years. It is not an experiment, but has stood the severest of all tests—that of practical use—which has demonstrated it to be the best method yet devised for the education of the busy working man.

¶ For purposes of ready reference and timely information when needed, it is believed that this series of handbooks will be found to meet every requirement.

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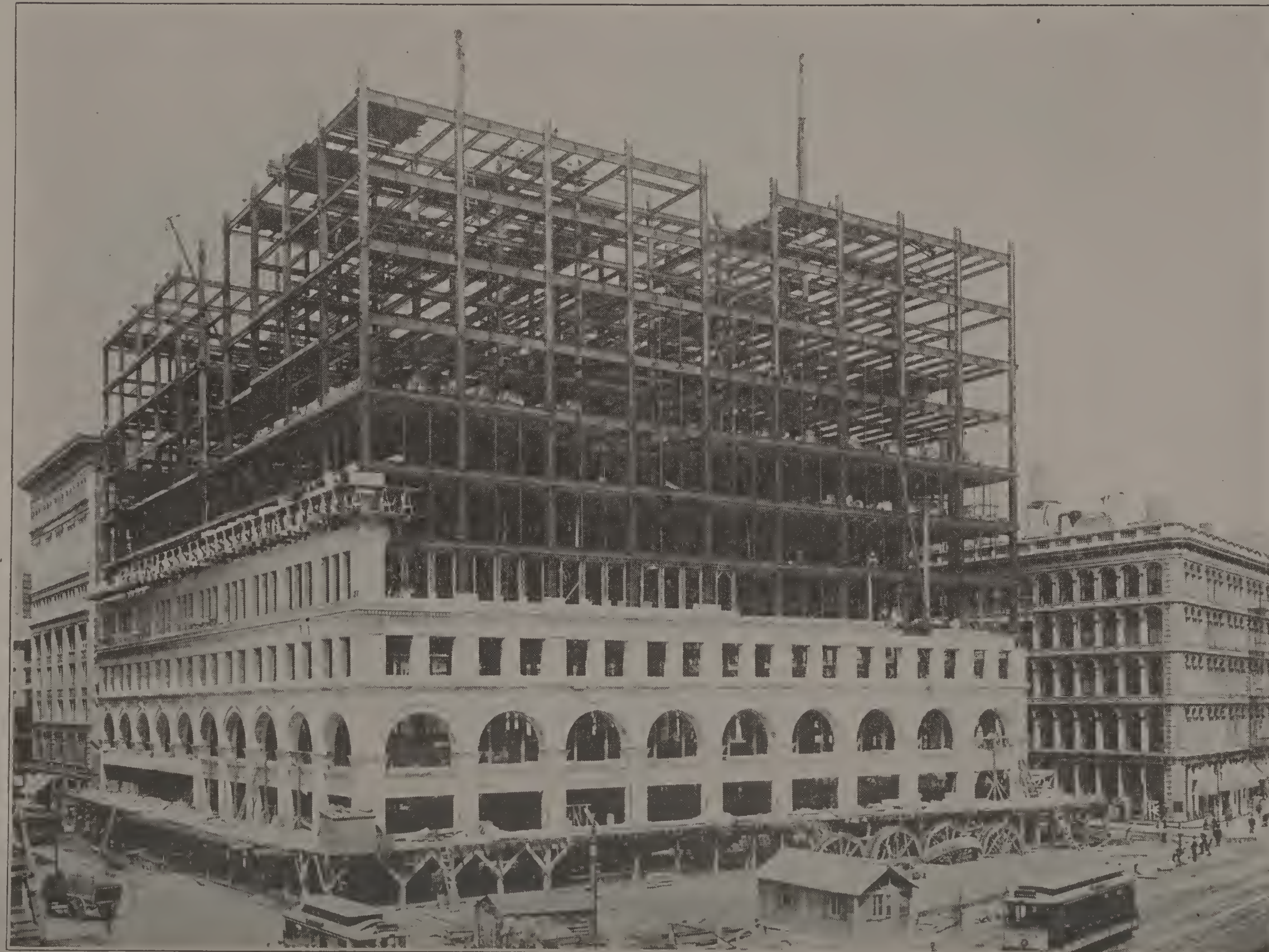

WANAMAKER BUILDING IN NEW YORK CITY, N. Y.
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Steel and Tile Construction Throughout

STEEL CONSTRUCTION.

PART I.

THE STRUCTURAL ELEMENTS OF A BUILDING.

From the structural point of view, a building consists of the following parts:

1. The foundations.
2. The enclosing walls.
3. The columns and bearing partitions.
4. The floors.
5. The roof.

If the building is very narrow, columns and bearing partitions may not be used, but the other four components are always present. Steel enters into the composition of the last four named parts to a greater or less extent in nearly every building, and these steel members are collectively called the framework of the building. Leaving the discussion of the subject of foundations until later, we shall consider briefly the component parts of the other divisions that may be said to constitute the elements of a building.

THE ENCLOSING WALLS.

Exterior walls, in general, are of five kinds:

1. Masonry walls of brick or stone, supporting their own weight and the adjacent floor and roof loads.
2. Masonry walls supporting their own weight, but no floor or roof loads.
3. Masonry walls not self-supporting.
4. Walls of iron, copper or other metal.
5. Walls of concrete.

Load-bearing Walls. Walls of the first class will be readily understood as regards their general characteristics, and will be treated more in detail under the heading "Building Laws and Specifications."

Self-supporting Walls. Walls of the second class are generally of brick or stone, and have contained in them steel elements carrying the floor and roof loads. These elements consist of vertical members spaced at intervals in the wall and called the **wall columns,** and, between them, horizontal members, generally at the floor levels and also over all openings. These members at the floor levels are called the **wall girders;** and those over the openings, the **lintels.** The wall girders carry the floor and roof loads to the columns, and so to the foundations. The lintels, in this class of wall, rest on the masonry and sometimes are omitted entirely, depending on the necessity of supporting the stone lintels, on the impracticability of turning brick arches, or on the necessity of relieving such arches of part of the load.

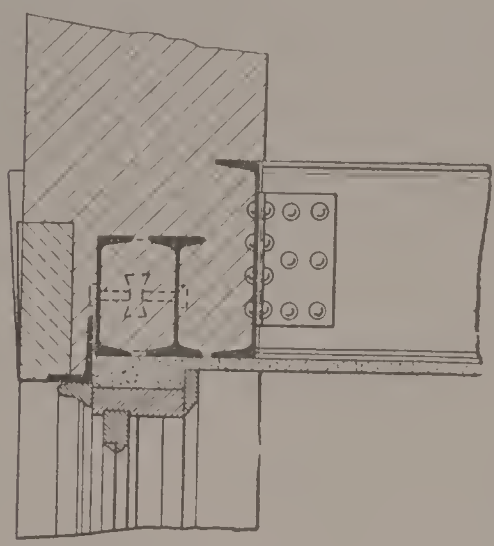


Fig 1

Fig. 1 shows a construction of this type. The particular form of section of the wall girders and of the lintels varies, of course, with the conditions; but the essential feature to be noted is that all loads are kept off the walls, except the weight of the masonry itself.

Curtain Walls. Walls of the third class differ from the preceding in that they themselves must be supported on the steel framework. The walls themselves may consist of brick, or of brick with stone or terra cotta trimmings or facings. The steel elements are the wall columns and wall girders, as before, and the horizontal members over the openings. These latter, instead of being called lintels, however, are called

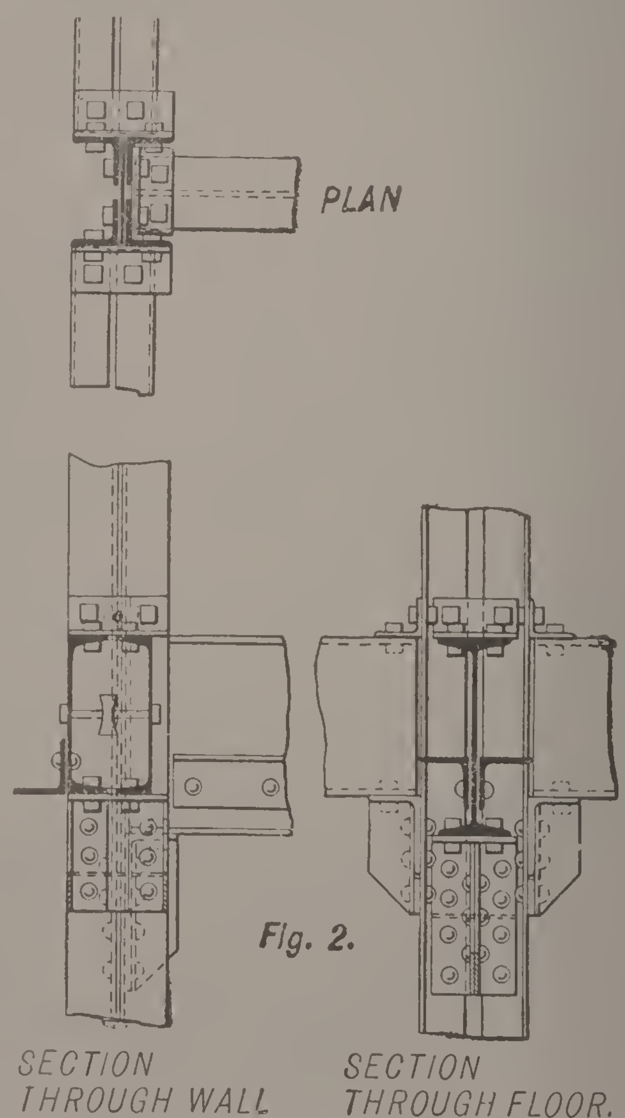
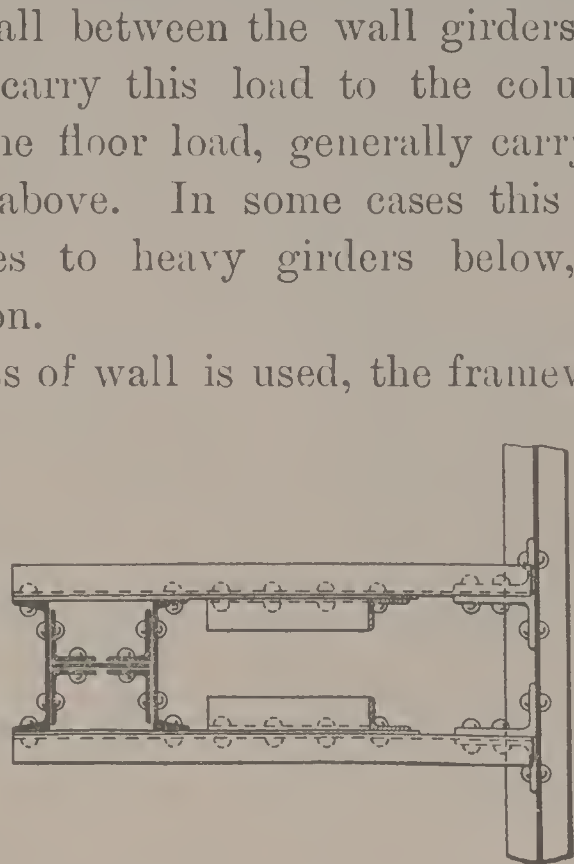


Fig. 2.

spandrel beams, since, instead of simply spanning the opening, they take all the load of the wall between the wall girders and the head of the opening, and carry this load to the columns. The wall girders, too, besides the floor load, generally carry the load of the wall for the story above. In some cases this wall is carried through several stories to heavy girders below, but such construction is not common.

In buildings where this class of wall is used, the framework, in addition to carrying the loads, must furnish a portion of the lateral stiffness to resist wind and other strains. This feature will be more particularly discussed under "High-Building Construction."



Figs. 2 and 3 show types of construction in this class.

Metal Walls. Walls of the fourth class are not commonly met with in what is termed fireproof construction, but are more generally used in buildings having their floors and roofs framed in whole or in part with wood. When they do occur, however, they come, structurally, into the previous class, as far as the elements of the framework necessary for the support of the floor and roof loads and their own weight are concerned.

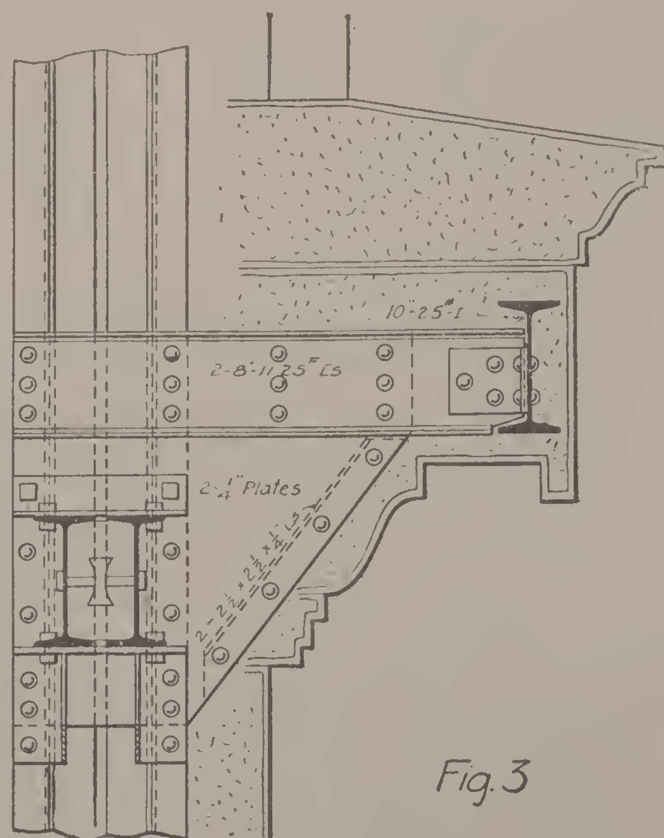


Fig. 3

The chief difference is in adapting the spandrel beams to the support of the particular covering used. Fig. 4 illustrates such construction. As before, the section of the wall girders varies in each case with the conditions, and the spandrel section varies even more.

In both of the two classes just described (curtain walls and

metal walls), no form of construction can be called standard. The only principle which the student should observe and follow is that the wall girders and spandrel beams must be so arranged and designed as to carry properly all floor and roof loads, to support and carry properly each part of the wall itself, and, further, to provide necessary stiffness to the building.

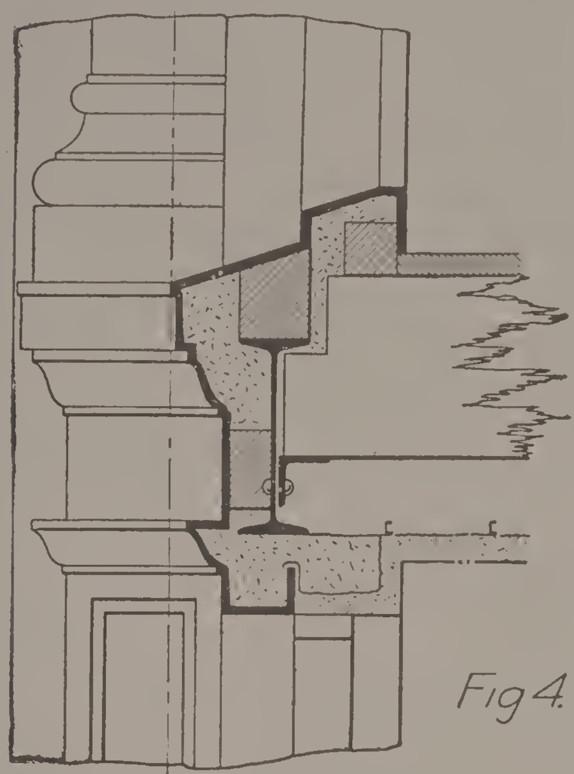


Fig 4

Concrete Walls. Walls of the fifth class are rarely met with at present, except in mills or manufacturing plants, and discussion of their features is accordingly reserved for the discussion on "Mill Buildings."

INTERIOR COLUMNS AND BEARING PARTITIONS.

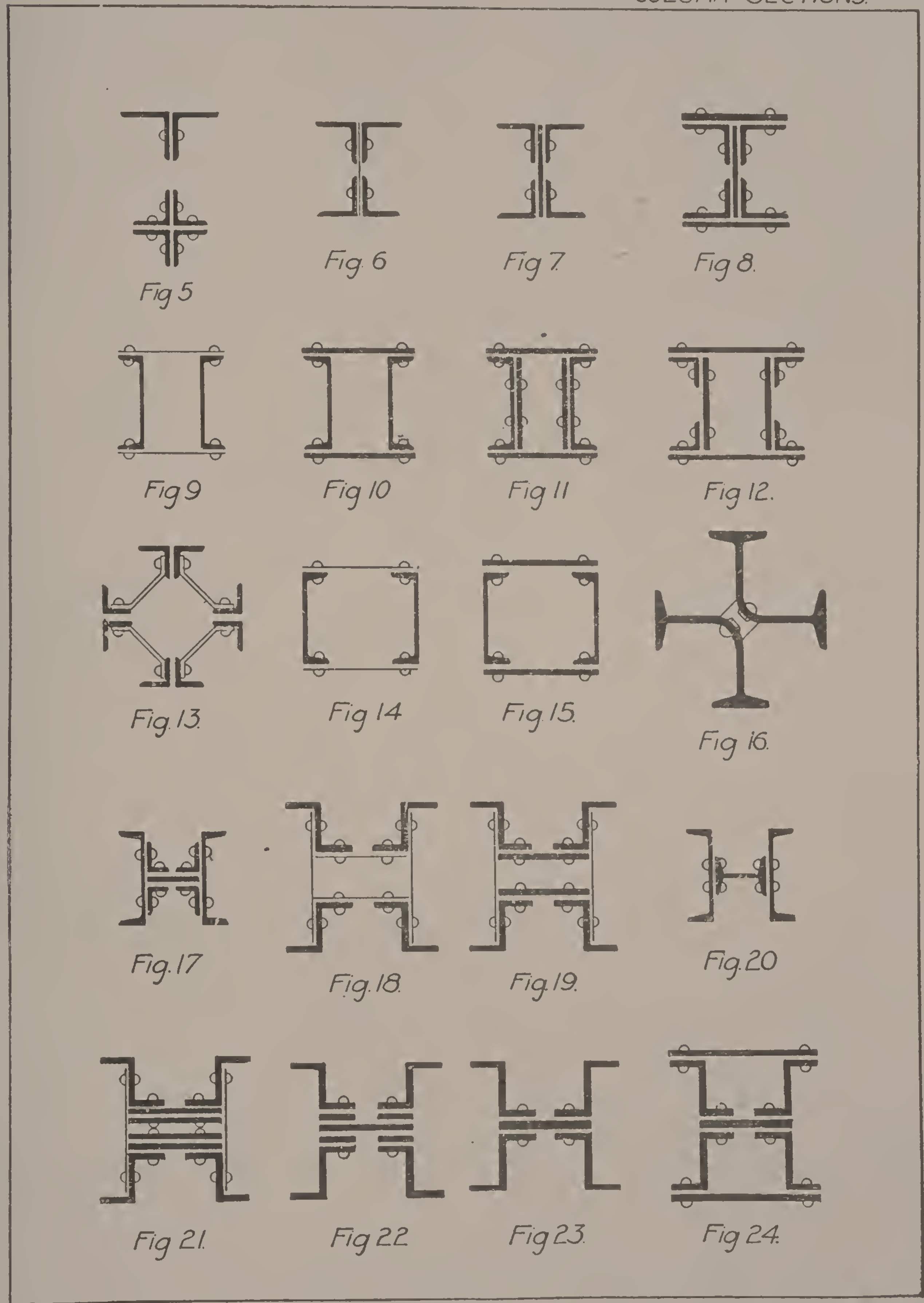
These are classed together because, either jointly or separately, they serve to carry to the foundations the portion of the loads not carried by the wall columns and exterior walls. When a partition takes these loads, it is invariably of brick. When partitions are of other materials, they are never designed to carry loads, but, on the contrary, form part of the load carried by the floors.*

The different forms of partitions that are not load-bearing will be considered under "Fireproofing."

Columns are the more common, and in general the exclusive, element of the framework that carries the loads within the walls to the foundations. There are two features to be considered in connection with them: (1) the load-bearing or metal shaft, and (2) its covering or casing. There are a variety of sections of columns, some of which are illustrated by Plate I. As in the case of forms of spandrel beams, no definite rule can be given for the use of any particular section to the exclusion of others. These will be described in detail under the heading "Columns."

* NOTE.—This statement refers to fireproof buildings only, and not to those framed with wood.

Plate I. COLUMN SECTIONS.



THE FLOORS.

The elements of the floor are:

1. The arch, which receives the load directly.
2. The beams, between which the arch springs.
3. The girders, carrying the beams.
4. The ceiling.

Arches. There are several different kinds of floor arch. In general, as to material of construction, they may be said to comprise the following types: brick, corrugated iron, porous terra cotta, hard tile, concrete, and concrete steel.

In office buildings and nearly all structures with a finished interior, some form of flat arch is used almost exclusively in order to avoid the necessity of furring down for a flat ceiling. In warehouses, stores, and other buildings carrying heavy loads, segmental arch construction is more frequent. All segmental arch constructions require tie rods passing through the webs of the beams at intervals of about five feet, to take the thrust of the arches. Tie rods are also required in flat arch construction, where the arch is made of separate blocks, but they are not generally used for flat arches of concrete slabs.

The subject of arches will be considered in detail under "Fireproofing."

Beams and Girders. All of the horizontal members that form the framing of the floor come under one or the other of these heads.

A **beam** carries no other element of the framework, and receives strictly the load of the arch or the partition or other portion of the structure which it is designed to carry.

A **girder** carries the end of one or more beams. It may at the same time receive direct load from the arch or partitions; but if it carries other elements of the framework it should be referred to as a beam.

Other uses of the terms "beam" and "girder" will be considered later.

THE ROOF.

A roof is essentially the same as a floor as regards the elements of construction. Its peculiar features are the pitch, openings for skylights, etc., support of pent houses, of tanks, etc.

The pitch in almost every case where a fireproof roof is used is very flat, generally a minimum of $\frac{3}{4}$ inch per foot and varying from that according to the requirements of the roof lines in each particular case.

The beams and girders usually follow the pitches of the finished surface of the roof, so that no additional grading on top of roof is necessary, except locally in order to form cradles around skylights and other obstructions, from the down-spout to the wall immediately back of it, and in a few places where the pitch of the roof necessarily changes between the bearings of beams. In general, however, the pitch of roof changes only at the ends of beams and girders.

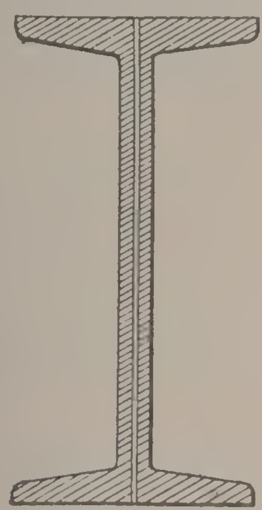


Fig. 25
BEAM

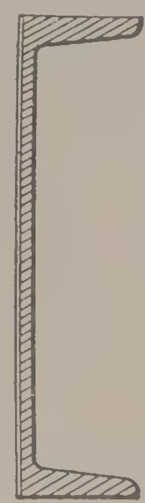


Fig. 26
CHANNEL

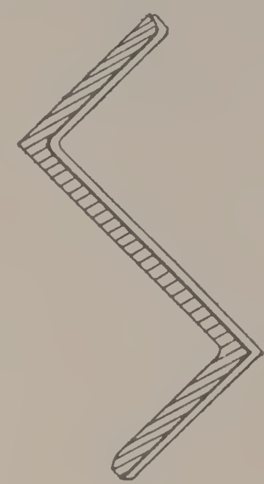


Fig. 27
ZEE

The pitching of beams and girders makes it necessary to furr down the ceiling, if this is to be left level, as it generally is. This is done by hanging from the beams a ceiling made either of tile or plastered wire lath on small angles or channels. See "Fireproofing" for illustrations of this.

Tanks and pent houses require special framing for their support, and all roof houses generally are constructed with a frame of light angles and tees.

USE OF HANDBOOKS ON STEEL.

The steel used in a building is in the form of single pieces, or combinations of one or more pieces, to which the general term "shapes" is applied. All shapes are made by rolling out the

rectangular prisms or ingots that come from the blast furnace. The following comprise nearly all the shapes rolled: Bars or Flats, Rounds, Half Rounds, Ovals, Flat Ovals, Plates, Angles, Tees, Zees, I Beams, Deck Beams, Channels, Trough Plates, Corrugated Plates, Buckled Plates. Illustrations of some of these are given in Figs. 25 to 35.

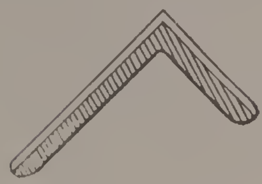


Fig 28
Unequal Leg Angle

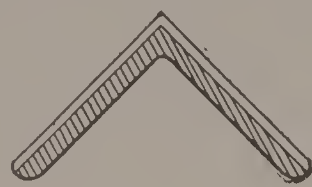


Fig 29
Equal Leg Angle

Method of Rolling. The processes of manufacture are practically identical in all mills; and the sizes of the different shapes are identical in all mills for nearly all sizes. Certain sizes are known as "standard," because they are rolled constantly by all mills. Certain other sizes not so commonly used are known as "special," and vary somewhat in the different mills. These distinctions will be explained in greater detail later on.

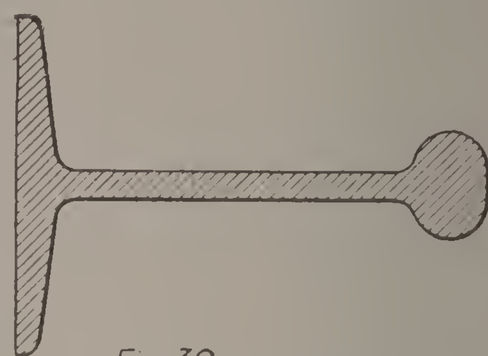


Fig 30
DECK BEAM

The process of rolling an I beam is in general as follows: The ingots are put into what are called "soaking pits" below ground, which are heated by natural gas. When white hot or at just the right temperature, they are taken out and passed several times through the first set of shaping rolls. These rolls are at first spread nearly the depth of the ingot. They are automatically lowered, however, as the ingot is passed through, and so reduce the thickness sufficiently to enable the piece to pass through the next set of rolls, which give it the general shape of the letter I, though it still retains much thickness, and is only partly formed. After being shaped sufficiently by these rolls, the piece is passed to the third or finishing set of rolls, where the final shaping takes place. The piece, still very hot, is then passed on by movable tables to circular saws, where it is cut into certain lengths. Each size and weight of beam or other shape requires a distinct set of rolls in order that the pieces may be given exactly

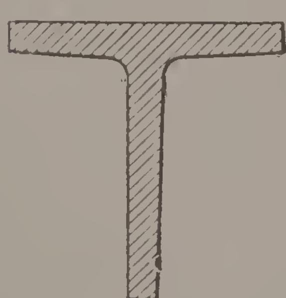


Fig. 31
TEE BAR

the required thickness and dimensions. Therefore, only one size and weight is rolled at a time, and all orders that have accumulated since the last rolling of this size are then rolled at once.

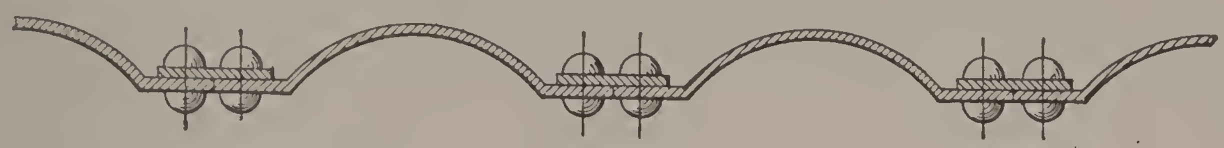


Fig 32

SECTION OF CORRUGATED PLATES FOR FLOORS.

The intervals of time that elapse between rollings of a given size vary considerably, being in some cases perhaps six weeks, and

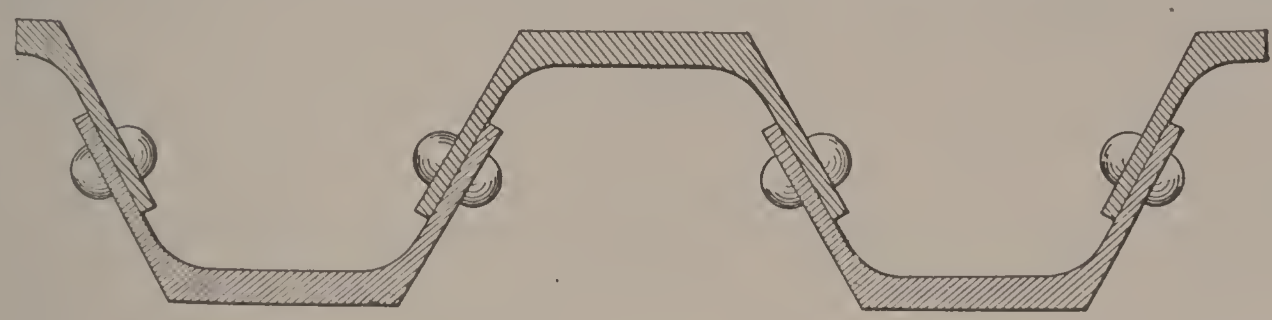


Fig 33.

SECTION OF TROUGH PLATES FOR FLOORS.

in other cases several months. Generally the larger sizes are rolled at one mill and the smaller sizes at another.

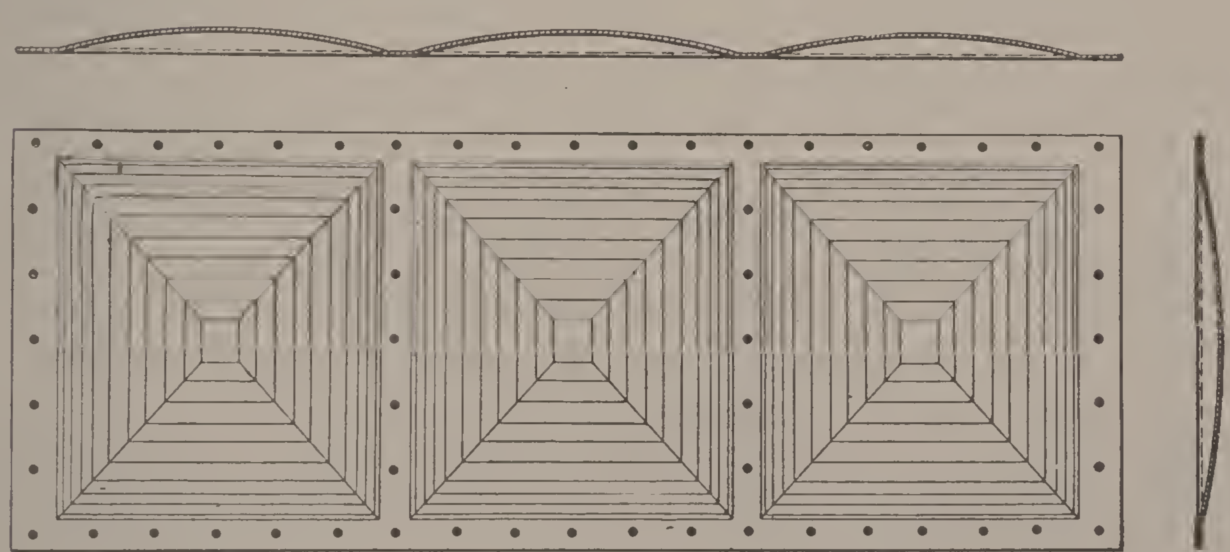


Fig. 34

SECTION OF BUCKLED PLATES FOR FLOORS.

Characteristics of Shapes. Having seen in general how shapes are formed, the student should now become thoroughly familiar with the features of each. **Beams and channels** consist of a thin plate-like portion, called the "web," and, outstanding at

each end of the web and at right angles to it, what are called "flanges." A beam has the shape of a letter I and is therefore called an I beam. A channel is like a letter I with the flanges on one side of the web omitted. The connection of flange to web is curved, and this curve is called the "fillet"; also, the inner side of a flange is beveled, and this bevel is in all sizes the same, *viz.*, $16\frac{2}{3}$ per cent with the outer side of the flange. A curve of varying radius connects the outer edge with the inner side of a flange. The distribution of metal in the heavier sections of a given shape is shown by the portion not cross hatched in Figs. 25 to 29. It will be seen therefore that for a given depth, the only difference in the different weights is in the thickness of webs and width of flanges.

Fig. 35
PLAIN ANGLE

The accompanying cuts, Fig. 36, shows the relations, radii of curvature, and other data which are standard for all beams.

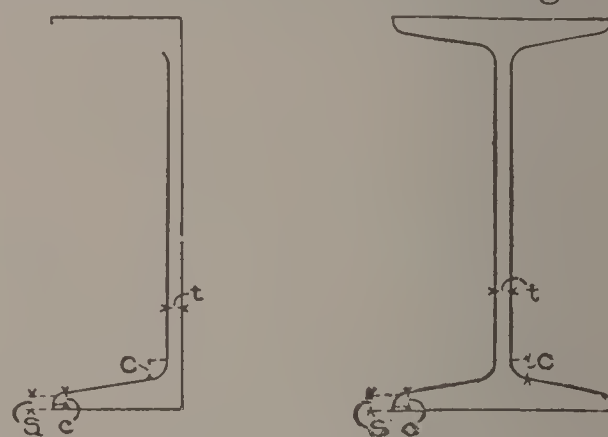


Fig. 36.

$c = .60$ minimum web
$C =$ minimum web $+ \frac{1}{10}$ inch
$s =$ thickness of web $= t$ minimum

NOTE. This applies for all channels and beams except 20-inch I and 24-inch I.

For 20-inch standard I,	$S = .55$ inch	$t = .50$ inch minimum
For 24-inch I,	$S = .60$ "	$t = .50$ " "
For 20-inch special I,	$S = .65$ "	$t = .60$ " "

The slope of flanges for all beams and channels is 2 inches per foot.

In tables V and VI, the weights printed in heavy type are those that are standard. The other weights are rolled by spreading the rolls of the standard size so as to give the required increase, and are known as special weights. These are not rolled so regularly, and are therefore in general more subject to delay in delivery.

The two parts of an **angle** are called "legs." These are in one class of equal length, and in another class of unequal length. Notice also the fillet and curve at outer edge. The method of increasing the weight is shown by the full lines. It will be seen,

therefore, that for an angle with certain size of legs the effect of increasing weight is to change slightly the length of legs, and to increase the thickness.

In case of angles, the distinction between "standard" and "special" applies, not to different weights and thicknesses of a given size as in the case of beams and channels, but to all weights of a given size as a whole, as will be seen from the tables on pages 36–7. Angles vary in all cases by $\frac{1}{16}$ inch in thickness between maximum and minimum thicknesses given in the tables. In the addition to the above special sizes of angles, there are certain

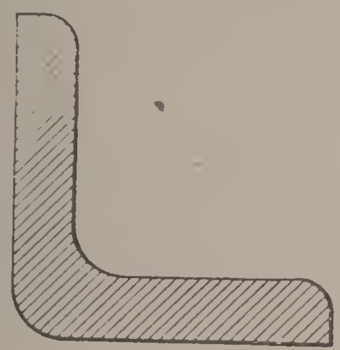


Fig 37
COVER ANGLE

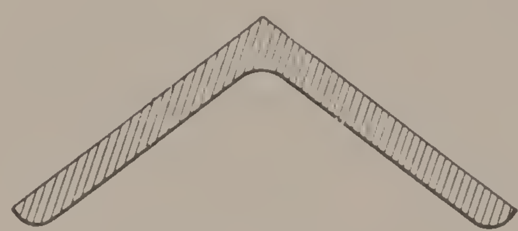


Fig 38
OBTUSE ANGLE

Fig 39
SAFE ANGLE

special shaped angles known as **square root angles, cover angles, obtuse angles,** and **safe angles.** These shapes are illustrated in Figs. 37, 38 and 39. Their uses, however, are limited to special classes of work.

The square root angles are used where it is necessary to eliminate the fillet. The cover angles are for use in splicing so that the covers will fit the fillets of the angles spliced. As the demand for such is limited in any particular piece of work, it is customary to plane off a regular angle. The other shapes are for special uses, as will be readily understood.

Bent plates are very commonly used in place of obtuse angles. None of the above can be obtained easily at the mills, and would be used only when it is not possible to adopt the regular shapes.

With the above explanation the student should be able to understand readily the features of the other shapes by carefully studying the cuts.

Plates are of two classes known as "sheared" plates and "universal mill" or "edged" plates. Plates up to 48 inches in width are in general universal mill plates. This term applies to

plates whose edges as well as surfaces are rolled, thus insuring uniform width. Plates above 48 inches in width have their edges sheared, and are known as sheared plates.

As already stated, there are various meanings of the terms "beam" and "girder," and it is very important to understand fully the distinctions. The definitions previously given are applied to the manner of loading.

"Beam" is also the term applied to the shape rolled in the form of the letter I, in distinction from the channel, as noted in the preceding paragraphs. An I beam may be used in a position which, from the definition given, fixes it as a girder in distinction from a beam; and in speaking of such a case, one should say that the girder consists of an I beam. In ordering the material, however, the shape should be referred to as an I beam and not as an I girder.

Similarly, a channel may be used in a position which, from the definition, would fix it as a beam. In referring to it, one should say that the beam consists of a channel; and in ordering material, it should be referred to as a channel and not as a beam.

The beam may in some cases be made of sections riveted together, and, in such cases, would be referred to, in ordering, as a riveted girder. Frequently, also, two beams bolted together are used, and are then called beam girders. It will be seen, therefore, that there are two distinct uses of these terms, beams and girders — the first depending on the manner of loading, and the second on the particular form of section of the member used. These two uses should never be confounded, as serious results might follow, especially in ordering material.

Uses of Sections. Each of the rolled sections has certain uses to which it is especially adapted, and for which it is most generally employed. **I beams** and **channels** are used principally as beams and girders to carry floors, roofs and walls. I beams are used to some extent also as columns, when the loads are relatively light. Channels are rarely used singly as columns; but they are used quite extensively in pairs latticed, and in combination with other shapes, to serve the purposes of columns. (For illustrations of such uses see Plate I, Page 7, showing column sections.)

Channels are also used to some extent in pairs latticed, or with plates across flanges, for the chords in trusses.

Angles are used most extensively in combination with other shapes to form columns, for members in trusses, and for the flanges of riveted girders. They are rarely used singly as columns except for light loads. As beams they are used only for very light loads, such as short lintels, ceilings, and roof purlins, when close spacing is necessary. They are used almost exclusively for the connections of beams and columns and of other members one with another, and for any position requiring a shelf for the support of other work.

The use of the angle is more varied than that of almost all other shapes, and it forms an essential part of nearly all riveted members.

Tees are rarely used in the construction of riveted members. Their principal uses are as beams of short spans and close spacing, where the loads are light and where a flange on each side of the center rib is necessary. Such instances occur in short lintels, ceilings, and certain cases of roofs, in skylights, pent houses and the like.

Zees are used extensively in columns, four zees being connected by a web plate or lattice bars; also to some extent in lintels and light purlins. They are seldom used except where it is desirable to have the flanges arranged in this way, and usually angles or tees can be used to equal advantage with less expense.

Plates are used as connecting members in nearly all riveted work, but rarely alone except as bearing surfaces on masonry, and in some cases as shelves built in and projecting from masonry walls to receive other members.

Buckled Plates and **Trough Plates** are used almost exclusively in bridge work for floors.

Corrugated Iron is used to a considerable extent in the siding and roofs of sheds and other buildings of a more or less temporary nature. Formerly it was used to some extent in fireproof floors as illustrated in "Fireproofing." This use, however, has almost entirely passed away.

Rods and **Bars** are used almost exclusively as tension members, for example, in trusses or as hangers.

Rules for Ordering. Material is never ordered simply from a schedule unless it is to be shipped plain, that is, merely cut to

length without any shop work upon it. If there is to be any working of the material other than cutting to length, such as punching, riveting, or framing, a shop drawing is invariably necessary. Descriptions and uses of shop drawings will be given later.

If the material is simply to be cut to length, however, a schedule is sufficient; and in such cases the following rules should be observed:

1. Never give both the thickness and weight per foot of a piece. Beams and channels are invariably ordered by the depth and weight per foot, as a 12-inch I beam $31\frac{1}{2}$ lbs. per foot, or a 10-inch channel 15 lbs. per foot.

Angles are almost invariably ordered by giving the dimensions of legs and the thickness, as a 6 in. $\times$ 6 in. $\times$ $\frac{1}{2}$ in. angle, or a 3 in. $\times 2\frac{1}{2}$ in. $\times$ $\frac{1}{4}$ in. angle.

Zees are generally ordered by giving dimensions and thickness, as a 3 in. $\times$ 3 in. $\times$ $\frac{3}{8}$ in. Z, or a 4 in. $\times$ 3 in. $\times$ $\frac{5}{16}$ in. Z. When unequal leg Z's are ordered, always give flange dimensions first.

In ordering tees, the dimensions and weight per foot are given, because the stem of a tee tapers. Thus a 3 in. $\times$ 3 in. 6.6-lb. T, or a $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. 9.2-lb. T. Here, as in the case of a Z, give flange dimensions first.

Plates are ordered by quoting width and thickness, as a 12 in. $\times$ $\frac{1}{2}$ in. plate. The same applies to bars and flats.

Square and round rods are ordered by giving dimensions of the cross-section, as a $\frac{7}{8}$-in. diameter rod, or a 2 in. $\times$ 2 in. rod.

2. All material, unless otherwise ordered, is subject to a standard variation in length of $\frac{3}{8}$ inch. That is, it may be $\frac{3}{8}$ inch over or under the specified length. If exact length is required, therefore, it is necessary to add after the specified length the word "exact."

3. If material is to be painted, the number of coats and kind of paint must be specified, as "Paint, one coat graphite."

4. Full shipping directions must be given, including the name of party or parties to whom order is to be billed, name of consignee, nearest railroad station, and route over which shipment is to be made.

5. Always avoid using special shapes and weights if time of delivery is any consideration, even at the expense of a little extra weight, unless special arrangement is made in advance as to the delivery which can surely be made. It is more important to avoid the delay that would hinder progress in all branches of the work on a building through waiting for a few pieces of steel, than it is to save a few pounds by the use of special shapes and weights.

USE OF TABLES.

Since all steel designs are dependent upon the use of the foregoing shapes, it will be seen that it is necessary to refer constantly to tables containing their dimensions and other characteristics called

"properties." This term "properties" covers all the characteristics which determine strength, and which are illustrated by the tables.

The different steel companies issue different books, but the properties for all standard shapes are practically the same.

Before proceeding to a discussion of the use of tables, a caution should be given for the future guidance of the student. There is always danger in using tables, diagrams, and formulæ prepared by someone else. The danger is from two sources: (1) the information given may not be correct; and (2) the person using the data may, through failure to understand fully the basis on which they were prepared, use them where they are not applicable.

As regards the first point, the more authoritative the book in which the information is given, the greater is the probability that it is correct. Not everything in print, however, is reliable.

The second point is even more important, because in the case of almost every table, diagram, or formula, there are certain limitations to its use, and certain cases to which, without a full understanding of these limitations, it is liable to be applied incorrectly.

From the outset the student should form the habit of investigating the derivation of tables and diagrams and the basis of formulæ in order that he may use them intelligently. The basis and application of the fundamental formulæ can be understood without necessarily retracing all the steps in their derivation. There are many special formulæ given which are simply modifications of the fundamental formulæ adapted to special cases, and such formulæ should never be used without tracing their derivation from the fundamental formulæ.

Safe Loads. Table I gives the total loads, uniformly distributed, which can be safely carried by the different sections of beams and channels for spans varying by one foot.

The manner in which the problem of the safe load will generally come up is:

Given a certain weight per linear foot of beam, and a certain span, to find the required size and weight of beam. In this case the total weight is obtained by multiplying the clear span by the weight per foot and adding the weight of the beam. As it is

TABLE I.

Safe Loads Uniformly Distributed for Standard and Special I-Beams and Channels, in Tons of 2,000 Pounds.

Distance between Supports in Feet	24" I 80 lbs.	Add for every lb. increase in weight	20" I 80 lbs.	20" I 65 lbs.	Add for every lb. increase in weight	18" I 55 lbs.	Add for every lb. increase in weight	15" I 80 lbs.	15" I 60 lbs.	15" I 42 lbs.	Add for every lb. increase in weight
12	77.33	.53	65.18	51.98	.44	39.29	.39	47.14	36.09	26.18	.33
13	71.38	.48	60.16	47.98	.40	36.27	.36	43.51	33.31	24.17	.30
14	66.28	.45	55.87	44.56	.37	33.68	.34	40.40	30.93	22.44	.28
15	61.86	.42	52.14	41.59	.35	31.43	.31	37.71	28.87	20.94	.26
16	58.00	.39	48.88	38.99	.33	29.47	.29	35.35	27.07	19.63	.24
17	54.58	.37	46.01	36.69	.31	27.74	.28	33.27	25.47	18.48	.23
18	51.56	.35	43.45	34.66	.29	26.19	.26	31.42	24.06	17.45	.22
19	48.84	.33	41.17	32.83	.28	24.82	.25	29.77	22.79	16.53	.21
20	46.40	.32	39.11	31.19	.26	23.58	.24	28.28	21.65	15.71	.20
21	44.19	.30	37.24	29.70	.25	22.45	.22	26.94	20.62	14.96	.19
22	42.18	.29	35.55	28.35	.24	21.43	.21	25.71	19.68	14.28	.18
23	40.35	.27	34.01	27.12	.23	20.50	.20	24.59	18.83	13.66	.17
24	38.67	.26	32.59	25.99	.22	19.65	.20	23.57	18.04	13.09	.16
25	37.12	.25	31.29	24.95	.21	18.86	.19	22.63	17.32	12.57	.16
26	35.69	.24	30.08	23.99	.20	18.14	.18	21.76	16.66	12.08	.15
27	34.37	.23	28.97	23.10	.19	17.46	.17	20.95	16.04	11.64	.14
28	33.14	.23	27.93	22.28	.19	16.84	.17	20.20	15.47	11.22	.14
29	32.00	.22	26.97	21.51	.18	16.26	.16	19.51	14.93	10.83	.13
30	30.93	.21	26.07	20.79	.17	15.72	.16	18.86	14.43	10.47	.13
31	29.94	.20	25.23	20.12	.17	15.21	.15	18.25	13.97	10.13	.13
32	29.00	.20	24.44	19.49	.16	14.73	.15	17.68	13.53	9.82	.12
33	28.12	.19	23.70	18.90	.16	14.29	.14	17.14	13.12	9.52	.12
34	27.29	.19	23.00	18.35	.15	13.87	.14	16.64	12.74	9.24	.11
35	26.51	.18	22.35	17.82	.15	13.47	.13	16.16	12.37	8.98	.11
36	25.78	.18	21.73	17.33	.15	13.10	.13	15.71	12.03	8.73	.11

Distance between Supports in Feet	12" I 40 lbs.	12" I 31.5 lbs.	Add for every lb. increase in weight	10" I 25 lbs.	Add for every lb. increase in weight	9" I 21 lbs.	Add for every lb. increase in weight	Distance between Supports in Feet	8" I 18 lbs.	Add for every lb. increase in weight
12	19.92	15.99	.26	10.85	.22	8.39	.20	5	15.17	.42
13	18.39	14.76	.24	10.02	.20	7.74	.18	6	12.64	.35
14	17.08	13.70	.23	9.30	.19	7.19	.17	7	10.84	.30
15	15.94	12.79	.21	8.68	.17	6.71	.16	8	9.48	.26
16	14.94	11.99	.20	8.14	.16	6.29	.15	9	8.43	.23
17	14.06	11.29	.19	7.66	.15	5.92	.14	10	7.59	.21
18	13.28	10.66	.18	7.24	.14	5.59	.13	11	6.90	.19
19	12.58	10.10	.17	6.86	.14	5.30	.12	12	6.32	.18
20	11.95	9.59	.16	6.51	.13	5.03	.12	13	5.83	.16
21	11.38	9.14	.15	6.20	.12	4.79	.11	14	5.42	.15
22	10.87	8.72	.14	5.92	.12	4.58	.11	15	5.06	.14
23	10.39	8.34	.14	5.66	.11	4.38	.10	16	4.74	.13
24	9.96	7.99	.13	5.43	.11	4.19	.10	17	4.46	.12
25	9.56	7.67	.13	5.21	.10	4.03	.09	18	4.21	.12
26	9.19	7.38	.12	5.01	.10	3.87	.09	19	3.99	.11
27	8.85	7.11	.12	4.82	.10	3.73	.09	20	3.79	.11
28	8.54	6.85	.11	4.65	.09	3.59	.08	21	3.61	.10
29	8.24	6.62	.11	4.49	.09	3.47	.08	....	......	....
30	7.97	6.40	.11	4.34	.09	3.36	.08	....	......	....

TABLE I—(Continued.)

Distance between Supports in Feet	7″ I 15 lbs.	Add for every lb. increase in weight	6″ I 12.25 lbs.	Add for every lb. increase in weight	5″ I 9.75 lbs.	Add for every lb. increase in weight	4″ I 7.5 lbs.	Add for every lb. increase in weight	3″ I 5.5 lbs.	Add for every lb. increase in weight
5	11.04	.36	7.75	.31	5.16	.26	3.18	.21	1.76	.16
6	9.20	.30	6.46	.26	4.30	.22	2.65	.18	1.47	.13
7	7.89	.26	5.54	.22	3.69	.19	2.27	.15	1.26	.11
8	6.90	.23	4.84	.19	3.23	.16	1.99	.13	1.10	.10
9	6.13	.20	4.31	.17	2.87	.14	1.77	12	0.98	.09
10	5.52	.18	3.88	.16	2.58	.13	1.59	.11	0.88	.08
11	5.02	.16	3.52	.14	2.35	.12	1.45	.10	0.80	.07
12	4.60	.15	3.23	.13	2.15	.11	1.33	.09	0.73	.07
13	4.25	.14	2.98	.12	1.98	.10	1.22	.08	0.68	.06
14	3.94	.13	2.77	.11	1.84	.09	1.14	.08	0.63	.06
15	3.68	.12	2.58	.10	1.72	.09	1.06	.07	0.59	.05
16	3.45	.11	2.42	.10	1.61	.08	0.99	.07	0.55	.05
17	3.25	.11	2.28	.09	1.52	.08	0.94	.06	0.52	.05
18	3.07	.10	2.15	.09	1.43	.07	0.88	.06	0.49	.04
19	2.91	.09	2.04	.08	1.36	.07	0.84	.06	0.46	04
20	2.76	.09	1.94	.08	1.29	.07	0.80	.05	0.44	.04
21	2.63	.09	1.85	.07	1.23	.06	0.76	.05	0.42	.04

Distance between Supports in Feet	15″ ⊏ 33 lbs.	Add for every lb. increase in weight	12″ ⊏ 20.5 lbs.	Add for every lb. increase in weight	10″ ⊏ 15 lbs.	Add for every lb. increase in weight	9″ ⊏ 13.25 lbs.	Add for every lb. increase in weight
10	22.23	.39	11.39	.32	7.14	.26	5.61	.24
11	20.20	.35	10.35	.29	6.49	.24	5.10	.21
12	18.52	.33	9.49	.26	5.95	.22	4.68	.20
13	17.10	.30	8.76	.24	5.49	.20	4.32	.18
14	15.87	.28	8.14	.23	5.10	.19	4.01	.17
15	14.82	.26	7.59	.21	4.76	.17	3.74	.16
16	13.89	.24	7.12	.20	4.46	.16	3.51	.15
17	13.07	.23	6.70	.18	4.20	.15	3.30	.14
18	12.35	.22	6.33	.18	3.96	.14	3.12	.13
19	11.70	.21	5.99	.17	3.76	.14	2.95	.12
20	11.11	.20	5.70	.16	3.57	.13	2.81	.12
21	10.58	.19	5.42	.15	3.40	.12	2.67	.11
22	10.10	.18	5.18	.14	3.24	.12	2.55	.11
23	9.66	.17	4.95	.14	3.10	.11	2.44	.10
24	9.26	.16	4.75	.13	2.97	.11	2.34	.10
25	8.89	.16	4.56	.13	2.85	.10	2.24	.09
26	8.55	.15	4.38	.12	2.74	.10	2.16	.09
27	8.23	.14	4.22	.12	2.64	.10	2.08	.09
28	7.94	.14	4.07	.11	2.55	.09	2.00	.08
29	7.66	.13	3.93	.11	2.46	.09	1.93	.08
30	7.41	.13	3.80	.11	2.38	.09	1.87	.08

Safe loads given include weight of beam. Maximum fibre stress 16,000 pounds per square inch.

TABLE I — (Concluded.)

Distance between Supports in Feet	8″ ⊏ 11.25 lbs.	Add for every lb. increase in weight	7″ ⊏ 9.75 lbs.	Add for every lb. increase in weight	6″ ⊏ 8 lbs.	Add for every lb. increase in weight	5″ ⊏ 6.5 lbs.	Add for every lb. increase in weight	4″ ⊏ 5.25 lbs.	Add for every lb. increase in weight	3″ ⊏ 4 lbs.	Add for every lb. increase in weight
5	8.61	.42	6.68	.36	4.62	.31	3.16	.26	2.02	.21	1.16	.16
6	7.18	.35	5.57	.30	3.85	.26	2.63	.22	1.68	.18	.97	.13
7	6.15	.30	4.77	.26	3.30	.22	2.26	.19	1.44	.15	.83	.11
8	5.38	.26	4.18	.23	2.89	.19	1.98	.16	1.26	.13	.73	.10
9	4.78	.23	3.71	.20	2.57	.17	1.76	.14	1.12	.12	.64	.09
10	4.31	.21	3.34	.18	2.31	.16	1.58	.13	1.01	.11	.58	.08
11	3.91	.19	3.04	.16	2.10	.14	1.44	.12	.92	.10	.53	.07
12	3.59	.18	2.78	.15	1.93	.13	1.32	.11	.84	.09	.48	.07
13	3.31	.16	2.57	.14	1.78	.12	1.22	.10	.78	.08	.45	.06
14	3.08	.15	2.39	.13	1.65	.11	1.13	.09	.72	.08	.41	.06
15	2.87	.14	2.23	.12	1.54	.10	1.05	.09	.67	.07	.39	.05
16	2.69	.13	2.09	.11	1.44	.10	.99	.08	.63	.07	.36	.05
17	2.53	.12	1.96	.11	1.36	.09	.93	.08	.59	.06	.34	.05
18	2.39	.11	1.86	.10	1.28	.09	.88	.07	.56	.06	.32	.04
19	2.27	.11	1.76	.09	1.22	.08	.83	.07	.53	.06	.31	.04
20	2.15	.11	1.67	.09	1.16	.08	.79	.07	.51	.05	.29	.04
21	2.05	.10	1.59	.09	1.10	.07	.75	.06	.48	.05	.28	.04
22	1.96	.10	1.52	.08	1.05	.07	.72	.06	.46	.05	.26	.04
23	1.87	.09	1.45	.08	1.00	.07	.69	.06	.44	.05	.25	.03
24	1.79	.09	1.39	.08	.96	.06	.66	.05	.42	.04	.24	.03
25	1.72	.08	1.34	.07	.92	.06	.63	.05	.40	.04	.23	.03

Safe loads given include weight of beam. Maximum fibre stress 16,000 lbs. per square inch.

necessary to know the size of the beam before its weight can be added, this operation must first be neglected, and the size provisionally determined from the tables showing what sections will carry the superimposed load. Then add the weight of the selected beam, and again refer to the table to see if the capacity has been exceeded by the addition of the weight of the beam. If it has, a different section must be taken.

It is important to note that there is in general a difference between the length of spans used in computing the total load carried and that used in the table. These tables are compiled from results given by the use of the regular beam formula, which

has been explained, and in this formula the length of span is the length between centers of bearings. It is this length which should be used in referring to the tables.

In some cases there would be practically no difference, as in the case of a beam framed between two steel girders. If, however, the beam were built into brick walls, the span used for computing total load would be the length between inside faces of walls, whereas the span used in tables would be from center to center of bearing plates.

Another point to be noticed in the use of these tables is that they are based on the supposition that the beam is supported by adjacent construction against **lateral deflection.** As will be more fully noted later on, long members under compression fail by deflecting sideways. In order, therefore, to be able to carry the full load indicated in these tables, the top flange of the beam or channel must be held against side deflection. This may be accomplished in a variety of ways. If the beam is in a floor or roof, the fireproof arches and the rods will generally provide the necessary support; or, if it is in a building not fireproof, the wood beams or the planking will also accomplish this. If, however, the beam was used in an unfinished attic, and the ceiling construction was at the bottom flange, leaving the rest of the beam exposed, the load must be reduced as indicated by the auxiliary table of proportionate loads. The load would also have to be reduced in the case of a beam carrying a wall with no cross framing at the level of the beam. It is, therefore, of the first importance to know exactly how the loads are carried by the beam, and in what relations other parts of the construction stand to the beam.

In practice, spans not exceeding twenty times the flange width are not considered to require side support.

In some cases there must be made still another modification of the loads indicated by these tables, and that is to provide against excessive **vertical deflection.** It is well known that all members loaded transversely will bend before they will break. In other words, any given load causes a certain amount of deflection. It is not practicable, however, to allow this deflection to be very great in structural members, because of the resulting vibration and because where there are plastered surfaces cracks will occur. It is

TABLE II.

Spacing of Standard I-Beams for Uniform Load of 100 Pounds per Square Foot. Proper Distance in Feet Center to Center of Beams.

Distance between Supports in Feet	24" I	20" I		18" I	15" I			12" I		10" I
	80 lbs.	80 lbs.	65 lbs.	55 lbs.	80 lbs.	60 lbs.	42 lbs.	40 lbs.	31.5 lbs.	25 lbs.
12	128.9	108.6	86.6	65.5	78.6	60.1	43.6	33.2	26.6	18.1
13	109.8	92.6	73.8	55.8	67.0	51.3	37.2	28.3	22.7	15.4
14	94.7	79.8	63.7	48.1	57.7	44.2	32.1	24.4	19.6	13.3
15	82.5	69.5	55.5	41.9	50.3	38.5	27.9	21.3	17.1	11.6
16	72.5	61.1	48.7	36.8	44.2	33.8	24.5	18.7	15.0	10.2
17	64.2	54.1	43.2	32.6	39.2	30.0	21.7	16.5	13.3	9.0
18	57.3	48.3	38.5	29.1	34.9	26.7	19.4	14.8	11.8	8.0
19	51.4	43.3	34.6	26.1	31.3	24.0	17.4	13.2	10.6	7.2
20	46.4	39.1	31.2	23.6	28.3	21.7	15.7	12.0	9.6	6.5
21	42.1	35.5	28.3	21.4	25.7	19.6	14.2	10.8	8.7	5.9
22	38.4	32.3	25.8	19.5	23.4	17.9	13.0	9.9	7.9	5.4
23	35.1	29.6	23.6	17.8	21.4	16.4	11.9	9.0	7.3	4.9
24	32.2	27.2	21.7	16.4	19.6	15.0	10.9	8.3	6.7	4.5
25	29.7	25.0	20.0	15.1	18.1	13.9	10.1	7.7	6.1	4.2
26	27.5	23.1	18.5	13.9	16.7	12.8	9.3	7.1	5.7	3.9
27	25.5	21.5	17.1	12.9	15.5	11.9	8.6	6.6	5.3	3.6
28	23.7	20.0	15.9	12.0	14.4	11.0	8.0	6.1	4.9	3.3
29	22.1	18.6	14.8	11.2	13.5	10.3	7.5	5.7	4.6	3.1
30	20.6	17.4	13.9	10.5	12.6	9.6	7.0	5.3	4.3	2.9

Distance between Supports in Feet	9" I	8" I	7" I	6" I	5" I	4" I	3" I
	21 lbs.	18 lbs.	15 lbs.	12.25 lbs.	9.75 lbs.	7.5 lbs.	5.5 lbs.
5	80.5	60.7	44.2	31.0	20.6	12.7	7.0
6	55.9	42.1	30.7	21.5	14.3	8.8	4.9
7	41.1	31.0	22.5	15.8	10.5	6.5	3.6
8	31.5	23.7	17.3	12.1	8.1	.5.0	2.8
9	24.9	18.7	13.6	9.6	6.4	3.9	2.2
10	20.1	15.2	11.1	7.8	5.2	3.2	1.8
11	16.6	12.5	9.1	6.4	4.3	2.6	1.5
12	14.0	10.5	7.7	5.4	3.6	2.2	1.2
13	11.9	9.0	6.5	4.6	3.1	1.9	1.0
14	10.3	7.7	5.6	4.0	2.6	1.6	0.9
15	9.0	6.7	4.9	3.4	2.3	1.4	
16	7.9	5.9	4.3	3.0	2.0	1.2	
17	7.0	5.3	3.8	2.7	1.8	1.1	
18	6.2	4.7	3.4	2.4	1.6	.98	
19	5.6	4.2	3.1	2.2	1.4		
20	5.0	3.8	2.8	1.9	1.3		
21	4.6	3.4	2.5	1.8	1.2		
22	3.8	3.1	2.3	1.6	1.1		

For load of 200 pounds per square foot, divide the spacing given by 2. Maximum fibre stress, 16,000 pounds per square inch.

TABLE II—(Concluded.)

Spacing of Standard I-Beams for Uniform Load of 150 Pounds per Square Foot. Proper Distance in Feet Center to Center of Beams.

Distance between Supports in Feet	24" I	20" I		18" I	15" I			12" I		10" I
	80 lbs.	80 lbs.	65 lbs.	55 lbs.	80 lbs.	60 lbs.	42 lbs.	40 lbs.	31.5 lbs.	25 lbs.
12	85.9	72.4	57.7	43.7	52.4	40.1	29.1	22.1	17.7	12.1
13	73.2	61.7	49.2	37.2	44.7	34.2	24.8	18.9	15.1	10.3
14	63.1	53.2	42.5	32.1	38.5	29.5	21.4	16.3	13.1	8.9
15	55.0	46.3	37.0	27.9	33.5	25.7	18.6	14.2	11.4	7.7
16	48.3	40.7	32.5	24.5	29.5	22.5	16.3	12.5	10.0	6.8
17	42.8	36.1	28.8	21.7	26.1	20.0	14.5	11.0	8.9	6.0
18	38.2	32.2	25.7	19.4	23.3	17.8	12.9	9.9	7.9	5.3
19	34.3	28.9	23.1	17.4	20.9	16.0	11.6	8.8	7.1	4.8
20	30.9	26.1	20.8	15.7	18.9	14.5	10.5	8.0	6.4	4.3
21	28.1	23.7	18.9	14.3	17.1	13.1	9.5	7.2	5.8	3.9
22	25.6	21.5	17.2	13.0	15.6	11.9	8.7	6.6	5.3	3.6
23	23.4	19.7	15.7	11.9	14.3	10.9	7.9	6.0	4.9	3.3
24	21.5	18.1	14.5	10.9	13.1	10.0	7.3	5.5	4.5	3.0
25	19.8	16.7	13.3	10.1	12.1	9.3	6.7	5.1	4.1	2.8
26	18.3	15.4	12.3	9.3	11.1	8.5	6.2	4.7	3.8	2.6
27	17.0	14.3	11.4	8.6	10.3	7.9	5.7	4.4	3.5	2.4
28	15.8	13.3	10.6	8.0	9.6	7.3	5.3	4.1	3.3	2.2
29	**14.7**	12.4	9.9	7.5	9.0	6.9	5.0	3.8	3.1	2.1
30	**13.7**	**11.6**	9.3	7.0	8.4	6.4	4.7	3.5	2.9	1.9

Distance between Supports in Feet	9" I	8" I	7" I	6" I	5" I	4" I	3" I
	21 lbs.	18 lbs.	15 lbs.	12.25 lbs.	9.75 lbs.	7.5 lbs.	5.5 lbs.
5	53.7	40.5	29.5	20.7	13.7	8.5	4.7
6	37.3	28.1	20.5	14.3	9.5	5.9	3.3
7	27.4	20.7	15.0	10.5	7.0	4.3	2.4
8	21.0	15.8	11.5	8.1	5.4	3.3	1.8
9	16.6	12.5	9.1	6.4	4.3	2.6	1.5
10	13.4	10.1	7.4	5.2	3.4	2.1	1.2
11	11.1	8.3	6.1	4.3	2.8	1.8	1.0
12	9.3	7.0	5.1	3.6	2.4	1.5	0.8
13	7.9	6.0	4.4	3.1	2.0	1.3	
14	6.9	5.2	3.8	2.6	1.8	1.1	
15	6.0	4.5	3.3	2.3	1.5	0.9	
16	5.2	4.0	2.9	2.0	1.4		
17	4.7	3.5	2.6	1.8	1.2		
18	4.1	3.1	2.3	1.6	1.1		
19	3.7	2.8	2.0	1.4	1.0		
20	3.4	2.5	1.8	1.3			
21	3.0	2.3	1.7	1.2			
22	2.8	2.1	1.5	1.1			

For load of 300 pounds per square foot, divide the spacing given by 2. Maximum fibre stress, 16,000 pounds per square inch.

not sufficient, therefore, merely to get a section strong enough to carry the given load, it must also be stiff enough not to deflect more than a certain proportion of its length under this load. It has been determined that a beam can deflect $\frac{1}{360}$ of its length, or $\frac{1}{30}$ of an inch per foot of length, without causing cracks in a plastered ceiling; and it is this criterion which is generally followed in determining the section required to meet the condition of safe deflection.

In Table I the loads above the heavy black line are the safe loads which can be carried without exceeding the above deflection. A beam may be used on spans longer than those above the black line; but in this case, in order not to exceed the safe deflection, the load indicated by the tables opposite this span must be reduced by the following rule:

Rule for Safe Loads above Spans Limited by Deflection. Divide the load given opposite the span corresponding to the length of beam by the corresponding span, and multiply by the span given just above the black line; or,

If S = the given span,
L = the tabular load for this span,
S_1 = the span just above the heavy black line,
L_1 = the required load,

then $L_1 = \frac{S_1 L}{S}$.

In cases where the depth of beam is not limited, comparison of different depths of beams should be made, and the one selected which proves the most economical.

Spacing of Beams. In many cases where the location of columns and spacing of beams are not fixed by certain features of design or construction, the problem arises in a form for which a table different from Table I is more useful. For instance, if the problem is to space the columns and beams to give the most economical sections to carry the given loads, Table II will be useful. This gives the spacing of beams for different spans to carry safely a load of 100 lbs. and 150 lbs. per square foot. By comparisons, therefore, of the different sections, spans, and spacing that may be used, the most economical section can be selected.

The above table is useful also when it is desired to know the

loading that a certain floor was designed to carry and when only the framing plan is at hand.

If other loads per square foot are used, the table can be modified by dividing the spacings given by the ratio of the required load to the indicated load of the table. The same modifications for lateral and vertical deflection must be made as in the preceding table.

In all cases where there is a choice between beams of different depths, it should be borne in mind that beams of greater depth than 15 inches cost an extra one-tenth of a cent per pound; this, therefore, affects their relative economy.

Deflection. As noted in preceding paragraphs, it is important to know what the vertical deflection of a shape will be under the loads and for the spans specified, as in the majority of cases the section cannot be selected from the tables of safe loads because of unequal loading or because some other shape is used. It is therefore necessary to be able to calculate from additional tables what the deflection will be.

The following formula can be readily used for this purpose. We shall first explain its derivation.

The general formula for the deflection of any shape supported at the ends and loaded uniformly is:

$$d = \frac{5\ Wl^3}{384\ EI}.$$

Where W is the total load, E the modulus of elasticity, and I the moment of inertia.

$$\frac{5}{384\ E} \text{ is a constant since } E = 29{,}000{,}000$$

$$W = pl, \text{ and } M = \tfrac{1}{8}\ pl^2 = \tfrac{1}{8}\ Wl$$

$$\text{and } \frac{M}{f} = \frac{I}{y} = \frac{Wl}{8 \times 16{,}000}; \text{ therefore } Wl = \frac{8 \times 16{,}000 \times I}{y}$$

if the beam is loaded up to its full capacity, and the fibre stress is taken at 16,000.

$$\text{Therefore } d = \frac{5l^2 \times 8 \times 16{,}000 \times I}{384\ E\ I\ y}$$

$$= \frac{.000575l^2}{y} \text{ (1), or, since } h = \text{depth of beam} = 2y,$$

$$= \frac{.000115l^2}{h} \text{ (2).}$$

In this formula l must be taken in inches.

From this general formula (1) a table in a number of different forms can be made. In Table III different values of l are substituted, so that the deflection in inches is obtained by taking the constant in the table corresponding to the given span, and dividing by the depth of the beam.

Another table could be made by substituting different values of h corresponding to different beams, and this would readily give for each beam the deflection by multiplying by the square of the span in inches.

If the fibre stress in the beam due to the loading was less than 16,000, the deflection would be obtained by multiplying the result given in the table by the ratio of given fibre stress to 16,000.

The formula (2) applies directly to beams and channels only. If, therefore, a table based on this formula is made, and it is desired to use it for determining the deflection of unsymmetrical shapes, such as angles, tees, etc., the coefficients given must be divided by twice the distance of the neutral axis from extreme fibre, since both numerator and denominator of (1) has been multiplied by 2.

If a beam had a center load, its deflection could be obtained from this table by multiplying by $\frac{8}{5}$, this being the ratio of the deflection of a beam supported at the ends and loaded with a center load, to that of a similar beam with the same total load uniformly distributed.

In the table of safe loads it will be noted that a heavy black line divides the capacities specified. This is to denote the limit of span beyond which the deflection of the beam, if loaded to its full capacity, would be likely to cause the ceiling to crack. This limit of span can be determined from the formulæ given above, as follows :

A deflection of $\frac{1}{360}$ of the span can be safely allowed without causing cracks. Substituting $\frac{l}{360}$ for d, therefore, we have

$$\frac{l}{360} = \frac{5l^2 \times 8 \times 16{,}000}{384 \text{ E } y}$$

and $l = 48.3\ y$

Making the substitutions of the value of y for different sized beams, gives limits agreeing with those in the Cambria Hand Book. The limits given in the Carnegie book are fixed arbitrarily at 20 times the depth of beam and some less than these.

Expressing the above formula in a different form, we have

$$f\,l = \frac{384\ \mathrm{E}\ y}{360 \times 5 \times 8}$$

$$\text{and } f\,\frac{l}{\mathrm{y}} = \frac{384\ \mathrm{E}}{360 \times 5 \times 8} = \mathrm{C}\ (\text{a constant}).$$

$$f\,\frac{l}{\mathrm{y}} = 773{,}333.$$

TABLE III.

Coefficients for Deflection in Inches for Cambria Shapes Used as Beams Subjected to Safe Loads Uniformly Distributed.

Distance between Supports in feet.	Coefficient for Fibre Stress of 16 000 lbs. per Square Inch.	Coefficient for Fibre Stress of 12 500 lbs. per Square Inch.	Distance between Supports in Feet.	Coefficient for Fibre Stress of 16 000 lbs. per Square Inch.	Coefficient for Fibre Stress of 12 500 lbs. per Square Inch.
L	H	H′	L	H	H′
4	.265	.207	23	8.756	6.841
5	.414	.323	24	9.534	7.448
6	.596	.466	25	10.345	8.082
7	.811	.634	26	11.189	8.741
8	1.059	.828	27	12.066	9.427
9	1.341	1.047	28	12.977	10.138
10	1.655	1.293	29	13.920	10.875
11	2.003	1.565	30	14.897	11.638
12	2.383	1.862	31	15.906	12.427
13	2.797	2.185	32	16.949	13.241
14	3.244	2.534	33	18.025	14.082
15	3.724	2.909	34	19.134	14.948
16	4.237	3.310	35	20.276	15.841
17	4.783	3.737	36	21.451	16.759
18	5.363	4.190	37	22.659	17.703
19	5.975	4.668	38	23.901	18.672
20	6.621	5.172	39	25.175	19.668
21	7.299	5.703	40	26.483	20.690
22	8.011	6.259			

This equation shows that if the table of properties is used to determine the capacity of a beam for a certain span which will be within the plaster limits of deflection, the product of the fibre strain and the span must be kept constant for a given depth of beam.

For example, if it is desired to know the fibre strain allowable for a 12-inch beam on an effective span of 30′–0″ (30 feet 0 inches) such that the plaster deflection will not be exceeded, we have

$$f = \frac{773{,}333 \times 6}{30 \times 12} = 12{,}888.$$

The formula can be more quickly used by comparison with the limiting span given by the table of safe loads. In the above case the limit of span for a 12-inch beam and a fibre strain of 16,000 lbs. is 24 feet; therefore the required

$$f = \frac{24}{30} \times 16{,}000$$
$$= 12{,}800$$

Lateral Deflection of Beams. When beams are used for long spans, and the construction is such that no support against side deflection is given, the beam will not safely carry the full load

TABLE IV.

Reduction in Values of Allowable Fibre Stress and Safe Loads for Shapes Used as Beams Due to Lateral Flexure.

Ratio of Span or Distance between Lateral Supports to Flange Width. $\frac{l}{b}$	Allowable Unit Stress for Direct Flexure in Extreme Fibre. p	Proportion of Tabular Safe Load to be Used.	Ratio of Span or Distance between Lateral Supports to Flange Width. $\frac{l}{b}$	Allowable Unit Stress for Direct Flexure in Extreme Fibre. p	Proportion of Tabular Safe Load to be Used
19.37	16000	1.0	65	7474	.47
20	15882	.97	70	6835	.43
25	14897	.93	75	6261	.39
30	13846	.87	80	5745	.36
35	12781	.80	85	5281	.33
40	11739	.73	90.	4865	.30
45	10746	.67	95	4595	.29
50	9818	.61	100	4154	.26
55	8963	.56	105	3850	.24
60	8182	.51	110	3576	.22

indicated by the table, and the allowable fibre stress in top flange must be reduced. If such a beam were to carry a load giving a fibre stress of 16,000 lbs. per square inch, the actual fibre stress in top flange would be greater than this, as the deflection sideways would tend to distort the top flange and thus cause the additional stresses.

The length of beam which it is customary to consider capable of safely carrying the full calculated load without support against lateral deflection, is twenty times the flange width. The reason for thus fixing upon twenty times the flange width may be seen from the following:

In any consideration of a reduction of stress in a compression member due to bending caused by its unsupported length, it is customary to use Gordon's formula for the safe stress in columns. This formula is:

$$f_c = \frac{f}{1 + \frac{l^2}{ar^2}}$$

For columns with fixed ends, $a = 36,000$. Now if we consider a 5-inch 9.75-lb. I, the moment of inertia about the neutral axis coincident with center line of web is $I' = 1.23$.

Since the moment of inertia of the web alone about this axis is inappreciable, the moment of inertia of each flange about this axis is $I'_f = .62$.

The area of the whole section = 2.87 sq. in.

$$\begin{aligned} \text{Web} &= \ \ .86 \\ \text{Area of flanges} &= 2.01 \text{ sq. in.} \\ \text{Area of one flange} &= 1.00 \quad \text{“} \\ \text{Therefore } r'^2_f &= \ \ .62 \\ r'_f &= \ \ .79 \end{aligned}$$

The width of flange for 5-in. beam $= b = 3.00$ in.

$$\text{Therefore } r'_f = \frac{b}{3.80}.$$

Tests on full-sized columns show that columns of length less than ninety times the radius of gyration bend little if any under their load. It is, therefore, generally customary to disregard the effect of bending for lengths less than 90 radii. If in the above we multiply, we have:

$$90\ r'_f = 23.7\ b$$

The assumption that with full fibre stress of 16,000 lbs. beams should be supported at distances not greater than twenty times the flange width, brings the limit under that of 90 radii.

Approximately the same result will be obtained if we assume the flange a rectangle and substitute 18,000 for f in Gordan's formula.

$$\text{Then } r^2 = \frac{b^2}{12}$$

$$\text{and } f_c = \frac{18,000}{1 - \frac{l^2}{3,000\,b^2}}$$

$$\text{and for } l = 20\ b$$

$$f_c = 15,900.$$

TABLE V.

Properties of I-Beams.

1	2	3	4	5	6	7	8	9
Section Index	Depth of Beam Inches	Weight per Foot Pounds	Area of Section Square Inches	Thickness of Web Inches	Width of Flange Inches	Mom. of Inertia Neutral Axis Perpendicular to Web at Center I	Mom. of Inertia Neutral Axis Coincident with Center Line of Web I'	Radius of Gyration Neutral Axis Perpendicular to Web at Center r
B 1	24	100.00	29.41	0.754	7.254	2380.3	48.56	9.00
		95.00	27.94	0.692	7.192	2309.6	47.10	9.09
		90.00	26.47	0.631	7.131	2239.1	45.70	9.20
		85.00	25.00	0.570	7.070	2168.6	44.35	9.31
		80.00	23.32	0.500	7.000	2087.9	42.86	9.46
B 2	20	100.00	29.41	0.884	7.284	1655.8	52.65	7.50
		95.00	27.94	0.810	7.210	1606.8	50.78	7.58
		90.00	26.47	0.737	7.137	1557.8	48.98	7.67
		85.00	25.00	0.663	7.063	1508.7	47.25	7.77
		80.00	23.73	0.600	7.000	1466.5	45.81	7.86
B 3	20	75.00	22.06	0.649	6.399	1268.9	30.25	7.58
		70.00	20.59	0.575	6.325	1219.9	29.04	7.70
		65.00	19.08	0.500	6.250	1169.6	27.86	7.83
B80	18	70.00	20.59	0.719	6.259	921.3	24.62	6.69
		65.00	19.12	0.637	6.177	881.5	23.47	6.79
		60.00	17.65	0.555	6.095	841.8	22.38	6.91
		55.00	15.93	0.460	6.000	795.6	21.19	7.07
B 4	15	100.00	29.41	1.184	6.774	900.5	50.98	5.53
		95.00	27.94	1.085	6.675	872.9	48.37	5.59
		90.00	26.47	0.987	6.577	845.4	45.91	5.65
		85.00	25.00	0.889	6.479	817.8	43.57	5.72
		80.00	23.81	0.810	6.400	795.5	41.76	5.78
B 5	15	75.00	22.06	0.882	6.292	691.2	30.68	5.60
		70.00	20.59	0.784	6.194	663.6	29.00	5.68
		65.00	19.12	0.686	6.096	636.0	27.42	5.77
		60.00	17.67	0.590	6.000	609.0	25.96	5.87
B 7	15	55.00	16.18	0.656	5.746	511.0	17.06	5.62
		50.00	14.71	0.558	5.648	483.4	16.04	5.73
		45.00	13.24	0.460	5.550	455.8	15.00	5.87
		42.00	12.48	0.410	5.500	441.7	14.62	5.95
B 8	12	55.00	16.18	0.822	5.612	321.0	17.46	4.45
		50.00	14.71	0.699	5.489	303.3	16.12	4.54
		45.00	13.24	0.576	5.366	285.7	14.89	4.65
		40.00	11.84	0.460	5.250	268.9	13.81	4.77
B 9	12	35.00	10.29	0.436	5.086	228.3	10.07	4.71
		31.50	9.26	0.350	5.000	215.8	9.50	4.83
B11	10	40.00	11.76	0.749	5.099	158.7	9.50	3.67
		35.00	10.29	0.602	4.952	146.4	8.52	3.77
		30.00	8.82	0.455	4.805	134.2	7.65	3.90
		25.00	7.37	0.310	4.660	122.1	6.89	4.07
B13	9	35.00	10.29	0.732	4.772	111.8	7.31	3.29
		30.00	8.82	0.569	4.609	101.9	6.42	3.40
		25.00	7.35	0.406	4.446	91.9	5.65	3.54
		21.00	6.31	0.290	4.330	84.9	5.16	3.67
B 15	8	25.50	7.50	0.541	4.271	68.4	4.75	3.02
		23.00	6.76	0.449	4.179	64.5	4.39	3.09
		20.50	6.03	0.357	4.087	60.6	4.07	3.17
		18.00	5.33	0.270	4.000	56.9	3.78	3.27
B 17	7	20.00	5.88	0.458	3.868	42.2	3.24	2.68
		17.50	5.15	0.353	3.763	39.2	2.94	2.76
		15.00	4.42	0.250	3.660	36.2	2.67	2.86
B 19	6	17.25	5.07	0.475	3.575	26.2	2.36	2.27
		14.75	4.34	0.352	3.452	24.0	2.09	2.35
		12.25	3.61	0.230	3.330	21.8	1.85	2.46
B 21	5	14.75	4.34	0.504	3.294	15.2	1.70	1.87
		12.25	3.60	0.357	3.147	13.6	1.45	1.94
		9.75	2.87	0.210	3.000	12.1	1.23	2.05
B 23	4	10.50	3.09	0.410	2.880	7.1	1.01	1.52
		9.50	2.79	0.337	2.807	6.7	0.93	1.55
		8.50	2.50	0.263	2.733	6.4	0.85	1.59
		7.50	2.21	0.190	2.660	6.0	0.77	1.64
B 77	3	7.50	2.21	0.361	2.521	2.9	0.60	1.15
		6.50	1.91	0.263	2.423	2.7	0.53	1.19
		5.50	1.63	0.170	2.330	2.5	0.46	1.23

TABLE V—(Continued.)

Properties of I-Beams.

10	11	12	13	14	15
Radius of Gyration Neutral Axis Coincident with Center Line of Web r'	Section Modulus Neutral Axis Perpendicular to Web at Center S'	Coefficient of Strength for Fiber Stress of 16,000 lbs. per sq. in. Used for Buildings C'	Coefficient of Strength for Fiber Stress of 12,500 lbs. per sq. in. Used for Bridges C'	Distance Center to Center Required to make Radii of Gyration equal (D)	Section Index
1.28	198.4	2115800	1653000	17.82	B 1
1.30	192.5	2052900	1603900	17.99	
1.31	186.6	1990300	1554900	18.21	
1.33	180.7	1927600	1505900	18.43	
1.36	**174.0**	**1855900**	**1449900**	**18.72**	
1.34	165.6	1766100	1379800	14.76	B 2
1.35	160.7	1713900	1339000	14.92	
1.36	155.8	1661600	1298100	15.10	
1.37	150.9	1609300	1257200	15.30	
1.39	**146.7**	**1564300**	**1222100**	**15.47**	
1.17	126.9	1353500	1057400	14.98	B 3
1.19	122.0	1301200	1016600	15.21	
1.21	**117.0**	**1247600**	**974700**	**15.47**	
1.09	102.4	1091900	853000	13.20	B 80
1.11	97.9	1044800	816200	13.40	
1.13	93.5	997700	779500	13.63	
1.15	**88.4**	**943000**	**736700**	**13.95**	
1.31	120.1	1280700	1000600	10.75	B 4
1.32	116.4	1241500	969900	10.86	
1.32	112.7	1202300	939300	10.99	
1.32	109.0	1163000	908600	11.13	
1.32	**106.1**	**1131300**	**883900**	**11.25**	
1.18	92.2	983000	768000	10.95	B 5
1.19	88.5	943800	737400	11.11	
1.20	84.8	904600	706700	11.29	
1.21	**81.2**	**866100**	**676600**	**11.49**	
1.02	68.1	726800	567800	11.05	B 7
1.04	64.5	687500	537100	11.27	
1.07	60.8	648200	506400	11.54	
1.08	**58.9**	**628300**	**490800**	**11.70**	
1.04	53.5	570600	445800	8.65	B 8
1.05	50.6	539200	421300	8.83	
1.06	47.6	507900	396800	9.06	
1.08	**44.8**	**478100**	**373500**	**9.29**	
0.99	38.0	405800	317000	9.21	B 9
1.01	**36.0**	**383700**	**299700**	**9.45**	
0.90	31.7	338500	264500	7.12	B 11
0.91	29.3	312400	244100	7.32	
0.93	26.8	286300	223600	7.57	
0.97	**24.4**	**260500**	**203500**	**7.91**	
0.84	24.8	265000	207000	6.36	B 13
0.85	22.6	241500	188700	7.58	
0.88	20.4	217900	170300	6.86	
0.90	**18.9**	**201300**	**157300**	**7.12**	
0.80	17.1	182500	142600	5.82	B 15
0.81	16.1	172000	134100	5.96	
0.82	15.1	161600	126200	6.12	
0.84	**14.2**	**151700**	**118500**	**6.32**	
0.74	12.1	128600	100400	5.15	B 17
0.76	11.2	119400	93300	5.31	
0.78	**10.4**	**110400**	**86300**	**5.50**	
0.68	8.7	93100	72800	4.33	B 19
0.69	8.0	85300	66600	4.49	
0.72	**7.3**	**77500**	**60500**	**4.70**	
0.63	6.1	64600	50500	......	B 21
0.63	5.4	58100	45400	......	
0.65	**4.8**	**51600**	**40300**	......	
0.57	3.6	38100	29800	......	B 23
0.58	3.4	36000	28100	......	
0.58	3.2	33900	26500	......	
0.59	**3.0**	**31800**	**24900**	......	
0.52	1.9	20700	1620	......	B 77
0.52	1.8	19100	15000	......	
0.53	**1.7**	**17600**	**13800**	......	

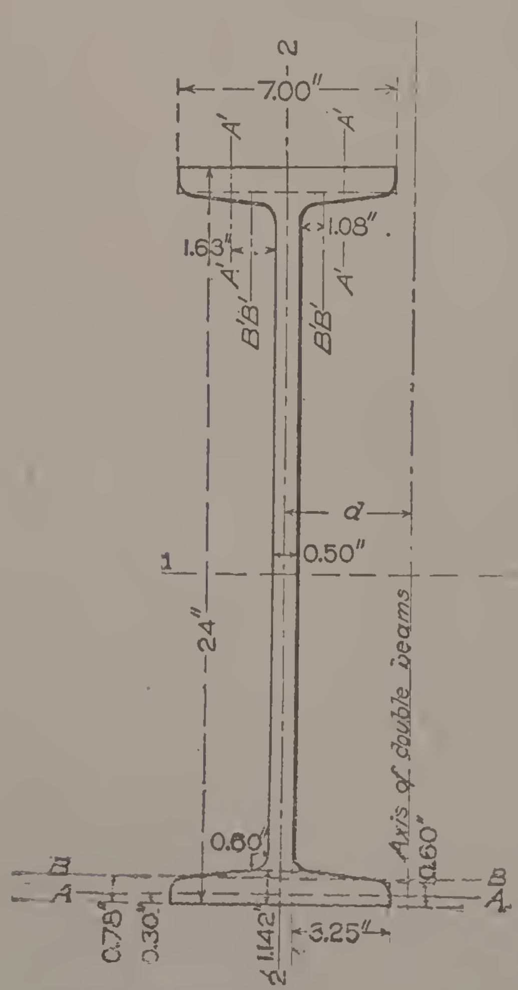
TABLE V — (Continued.)
Properties of Carnegie Trough Plates.

Section Index	Size Inches	Thickness Inches	Weight per Foot Pounds	Area of Section Square Inches	Moment of Inertia Neutral Axis Parallel to Length I	Section Modulus Axis as Before S	Radius of Gyration Axis as Before r
M 10	9½x3¾	½	16.3	4.8	3.68	1.38	0.91
M 11	9½x3¾	$\frac{9}{16}$	18.0	5.3	4.13	1.57	0.91
M 12	9½x3¾	⅝	19.7	5.8	4.57	1.77	0.90
M 13	9½x3¾	$\frac{11}{16}$	21.4	6.3	5.02	1 96	0.90
M 14	9½x3¾	¾	23 2	6.8	5.46	2.15	0.90

TABLE V — (Concluded.)
Properties of Carnegie Corrugated Plates.

Section Index	Size Inches	Thickness Inches	Weight per Foot Pounds	Area of Section Square Inch	Moment of Inertia Neutral Axis Parallel to Length I	Section Modulus Axis as before S	r Radius of Gyration Axis as before
M 30	8¾x1½	¼	8.1	2.4	0.64	0.80	0.52
M 31	8¾x1$\frac{9}{16}$	$\frac{5}{16}$	10.1	3.0	0.95	1.13	0.57
M 32	8¾x1⅝	⅜	12.0	3.5	1.25	1.42	0.62
M 33	12$\frac{3}{16}$x2¼	⅜	17.75	5.2	4.79	3.33	0.96
M 34	12$\frac{3}{16}$x2$\frac{13}{16}$	$\frac{7}{16}$	20.71	6.1	5.81	3.90	0.98
M 35	12$\frac{3}{16}$x2⅞	½	23.67	7.0	6.82	4.46	0.99

Table IV gives values to use for fibre stress, and proportions of full tabular load to use for different ratios of length and width of flange.



BEAM SECTION

Tables V, VI, VII, and VIII give the properties of the minimum and maximum sizes of the different shapes. These tables are for use in choosing sections to meet the requirements of design, and will be explained in detail in the pages that treat of design of members in which these shapes are used.

These different functions can all be calculated quite readily, and it is important that the student should understand how these are obtained. For this purpose the functions of a 24-inch 80-lb. beam will be worked out. The section of the beam is here shown.

AREA.

$$\text{Web} = (24 - 2.284) \times .50 \qquad\qquad = 10.858$$

$$\text{Flanges} = \frac{1.142 + .60}{2} \times 3.25 \times 4 = 11.323$$

$$1.142 \times .50 \times 2 \qquad = \underline{1.142} \qquad \frac{12.465}{23.323}$$

It will be noticed that the areas of fillets and the roundings of outer edges are disregarded. These closely offset each other.

WEIGHT PER FOOT.

Since a cubic foot of steel weighs 490 lbs., the weight per foot of a 24-inch beam should be:

$$\frac{23.323 \times 12}{1,728} \times 490 = 79.331 \text{ lbs.}$$

MOMENT OF INERTIA ABOUT AXIS 1 — 1.

I of web (taken to outside of flanges) $= \frac{1}{12} \times \frac{1}{2} \times 24^3 = 576.$

I′ of flange about an axis through center of gravity of each component element.

$$\text{Axis A A} = \frac{1}{12} \times 3.25 \times .60^3 \times 4 = .234$$

$$\text{Axis B B} = \frac{1}{36} \times 3.25\,(1.142 - .60)^3 = \frac{.057}{.291}$$

I of flanges about axis $1 - 1 = I' + A \times 4d^2$.

Where A = area of flanges, and d = distance from center of gravity of flange to axis $1 - 1$, as in the above, the flanges being divided into two figures, the d in each case will be the distance from $1 - 1$ to the center of gravity of that figure.

$$I = 3.25 \times .60 \times 11.70^2 \times 4 = 1,067.752$$

$$3.25 \times .271 \times 11.22^2 \times 4 = \underline{443.505} \qquad 1,511.548$$

$$\underline{576.}$$

$$2,087.548$$

TABLE VI.

Properties of Channels.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Section Index	Depth of Channel Inches	Weight per Foot Pounds	Area of Section Square Inches	Thickness of Web Inches	Width of Flange Inches	Mom. of Inertia Neutral Axis Perpendicular to Web at Center I	Mom. of Inertia Neutral Axis Parallel with Center Line of Web I′	Radius of Gyration Neutral Axis Perpendicular to Web at Center r	Radius of Gyration Neutral Axis Parallel with Center Line of Web r	Section Modulus Neutral Axis Perpendicular to Web at Center S	Coefficient of Strength for Fiber Stress of 16,000 lbs. per sq. in. Used for Buildings C	Coefficient of Strength for Fiber Stress of 12,500 lbs. per sq. in. Used for Bridges C	Distance Required to make Radii of Gyration Equal D	Distance of Center of Gravity from Outside of Web x	Section Index
C 1	15	55.00	16.18	0.818	3.818	430.2	12.19	5.16	.868	57.4	611900	478000	8.53	0.823	C 1
		50.00	14.71	0.720	3.720	402.7	11.22	5.23	.873	53.7	572700	447400	8.71	0.803	
		45.00	13.24	0.622	3.622	375.1	10.29	5.32	.882	50.0	533500	416800	8.92	0.788	
		40.00	11.76	0.524	3.524	347.5	9.39	5.43	.893	46.3	494200	386100	9.15	0.783	
		35.00	10.29	0.426	3.426	320.0	8.48	5.58	.908	42.7	455000	355500	9.43	0.789	
		33.00	9.90	0.400	3.400	312.6	8.23	5.62	.912	41.7	444500	347300	9.50	0.794	
C 2	12	40.00	11.76	0.758	3.418	197.0	6.63	4.09	.751	32.8	350200	273600	6.60	0.722	C 2
		35.00	10.29	0.636	3.296	179.3	5.90	4.17	.757	29.9	318800	249100	6.81	0.694	
		30.00	8.82	0.513	3.173	161.7	5.21	4.28	.768	26.9	287400	224500	7.07	0.677	
		25.00	7.35	0.390	3.050	141.0	4.53	4.43	.785	24.0	256100	200000	7.36	0.678	
		20.50	6.03	0.280	2.940	128.1	3.91	4.61	.805	21.4	227800	178000	7.67	0.704	
C 3	10	35.00	10.29	0.823	3.183	115.5	4.66	3.35	.672	23.1	246400	192500	5.17	0.695	C 3
		30.00	8.82	0.676	3.036	103.2	3.90	3.42	.672	20.6	220300	172100	5.40	0.651	
		25.00	7.35	0.529	2.889	91.0	3.40	3.52	.680	18.2	194100	151700	5.67	0.620	
		20.00	5.88	0.382	2.742	78.7	2.85	3.66	.696	15.7	168000	131200	5.97	0.609	
		15.00	4:46	0.240	2.600	66.9	2.30	3.87	.718	13.4	142700	111500	6.33	0.639	
C 4	9	25.00	7.35	0.615	2.815	70.7	2.98	3.10	.637	15.7	167600	130900	4.84	0.615	C 4
		20.00	5.88	0.452	2.652	60.8	2.45	3.21	.646	13.5	144100	112600	5.12	0.585	
		15.00	4.41	0.288	2.488	50.9	1.95	3.40	.665	11.3	120500	94200	5.49	0.590	
		13.25	3.89	0.230	2.430	47.3	1.77	3.49	.674	10.5	112200	87600	5.63	0.607	
C 5	8	21.25	6.25	0.582	2.622	47.8	2.25	2.77	.600	11.9	127400	99500	4.23	0.587	C 5
		18.75	5.51	0.490	2.530	43.8	2.01	2.82	.603	11.0	116900	91300	4.38	0.567	
		16.25	4.78	0.399	2.439	39.9	1.78	2.89	.610	10.0	106400	83200	4.54	0.556	
		13.75	4.04	0.307	2.347	36.0	1.55	2.98	.619	9.0	96000	75000	4.72	0.557	
		11.25	3.35	0.220	2.260	32.3	1.33	3.11	.630	8.1	86100	67300	4.94	0.576	
C 6	7	19.75	5.81	0.633	2.513	33.2	1.85	2.39	.565	9.5	101100	79000	3.48	0.583	C 6
		17.25	5.07	0.528	2.408	30.2	1.62	2.44	.564	8.6	92000	71800	3.64	0.555	
		14.75	4.31	·0.423	2.303	27.2	1.40	2.50	.568	7.8	82800	64700	3.80	0.535	
		12.25	3.60	0.318	2.198	24.2	1.19	2.59	.575	6.9	73700	57500	3.99	0.528	
		9.75	2.85	0.210	2.090	21.1	0.98	2.72	.586	6.0	66800	52200	4.22	0.546	
C 7	6	15.50	4.56	0.563	2.283	19.5	1.28	2.07	.529	6.5	69500	54300	2.91	0.546	C 7
		13.00	3.82	0.410	2.160	17.3	1.07	2.13	.529	5.8	61600	48100	3.09	0.517	
		10.50	3.09	0.318	2.038	15.1	0.88	2.21	.534	5.0	53800	42000	3.28	0.503	
		8.00	2.38	0.200	1.920	13.0	0.70	2.34	542	4.3	46200	36100	3.52	0.517	
C 8	5	11.50	3.38	0.477	2.037	10.4	0.82	1.75	.493	4.2	44400	34700	2.34	0.508	C 8
		9.00	2.65	0.330	1.890	8.9	0.64	1.83	.493	3.5	37900	29600	2.56	0.481	
		6.50	1.95	0.190	1.750	7.4	0.48	1.95	.498	3.0	31600	24700	2.79	0.489	
C 9	4	7.25	2.13	0.325	1.725	4.6	0.41	1.46	.455	2.3	24400	19000	1.85	0.463	C 9
		6.25	1.84	0.252	1.652	4.2	0.38	1.51	.454	2.1	22300	17400	1.96	0.458	
		5.25	1.55	0.180	1.580	3.8	0.32	1.56	.453	1.9	20200	15800	2.06	0.464	
C 72	3	6.00	1.76	0.362	1.602	2.1	0.31	1.08	.421	1.4	14700	11500	1.07	0.459	C 72
		5.00	1.47	0.261	1.501	1.8	0.25	1.12	.415	1.2	13100	10300	1.19	0.443	
		4.00	1.19	0.170	1.410	1.6	0.20	1.17	.409	1.1	11600	9100	1.31	0.443	

MOMENT OF INERTIA ABOUT AXIS 2—2.

Web — $\frac{1}{12} \times 24 \times .50^3 =$ .250

Flanges axis A′ A′ $= \frac{1}{12} \times .60 \times 3.25^3 \times 4 =$ 6.866

For axis 2-2 $= + 3.25 \times .60 \times (1.623 + .25)^2 \times 4 = 27.362$

Flange axis B′ B′ $= \frac{1}{36} \times .542 \times 3.25^3 \times 4 \quad = 2.060$

For axis 2-2 $= + .542 \times 1.63 \times (1.08 + .25)^2 \times 4 = \frac{6.250 \quad 42.538}{42.788}$

Other methods of computing the moments of inertia would perhaps bring a result even closer to the values given in the tables, which are taken from the Carnegie Handbook, although the values vary a little in the different books for identical sections.

RADIUS OF GYRATION. By definition, the radius of gyration is equal to the square root of the quotient of the moment of inertia divided by the area of the section; therefore, if $r_{1\text{-}1}$ and $r_{2\text{-}2}$ correspond to radii of gyration about the axes 1-1 and 2-2 respectively,

$$r_{1\text{-}1} = \sqrt{\frac{2{,}087.548}{23.323}} = 9.46$$

$$r_{2\text{-}2} = \sqrt{\frac{42.538}{23.323}} = 1.35$$

SECTION MODULUS. In the calculation of stresses in beams, the formula used is:

$$M = \frac{f\,I}{y};$$

$$\text{or,} \quad \frac{M}{f} = \frac{I}{y}.$$

The proper section of beam could be determined by this formula, using the moment of inertia, the distance from the neutral axis to the extreme fibre, and the allowable fibre stress. It is more convenient, however, to have the constant $\frac{I}{y}$ expressed in the tables, and this constant is called the "Section Modulus."

In the above case, therefore,

$$S_{1\text{-}1} = \frac{I}{y} = \frac{2{,}087.548}{12} = 173.96$$

TABLE VII.

Properties of Standard and Special Angles.

Angles with Equal Legs.

1	2	3	4	5	6	7	8	9	10
Section Index	Size Inches	Thickness, Inches	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity from Back of Flange, Inches	Moment of Inertia, Neutral Axis through Center of Gravity Parallel to Flange I	Section Modulus, Neutral Axis as before S	Radius of Gyration, Neutral Axis as before r	Least Radius of Gyration, Neutral Axis through Center of Gravity at Angle of 45 Degrees to Flanges r
A113	8 x8	1⅛	56.9	16.73	2.41	97.97	17.53	2.42	1.55
A112	8 x8	1 1/16	54.0	15.87	2.39	93.53	16.67	2.43	1.56
A111	8 x8	1	51.0	15.00	2.37	88.98	15.80	2.44	1.56
A110	8 x8	15/16	48.1	14.12	2.34	84.33	14.91	2.44	1.56
A109	8 x8	⅞	45.0	13.23	2.32	79.58	14.01	2.45	1.57
A108	8 x8	13/16	42.0	12.34	2.30	74.71	13.11	2.46	1.57
A107	8 x8	¾	38.9	11.44	2.28	69.74	12.18	2.47	1.57
A106	8 x8	11/16	35.8	10.53	2.25	64.64	11.25	2.48	1.58
A105	8 x8	⅝	32.7	9.61	2.23	59.42	10.30	2.49	1.58
A104	8 x8	9/16	29.6	8.68	2.21	54.09	9.34	2.50	1.58
A103	8 x8	½	26.4	7.75	2.19	48.63	8.37	2.50	1.58
A 86	6 x6	1	37.4	11.00	1.86	35.46	8.57	1.80	1.16
A 87	6 x6	15/16	35.3	10.37	1.84	33.72	8.11	1.80	1.16
A 1	6 x6	⅞	33.1	9.74	1.82	31.92	7.64	1.81	1.17
A 2	6 x6	13/16	31.0	9.09	1.80	30.06	7.15	1.82	1.17
A 3	6 x6	¾	28.7	8.44	1.78	28.15	6.66	1.83	1.17
A 4	6 x6	11/16	26.5	7.78	1.75	26.19	6.17	1.83	1.17
A 5	6 x6	⅝	24.2	7.11	1.73	24.16	5.66	1.84	1.18
A 6	6 x6	9/16	21.9	6.43	1.71	22.07	5.14	1.85	1.18
A 7	6 x6	½	19.6	5.75	1.68	19.91	4.61	1.86	1.18
A 8	6 x6	7/16	17.2	5.06	1.66	17.68	4.07	1.87	1.19
A 88	6 x6	⅜	14.9	4.36	1.64	15.39	3.53	1.88	1.19
*A 94	5 x5	1	30.6	9.00	1.61	19.64	5.80	1.48	0.96
*A 95	5 x5	15/16	28.9	8.50	1.59	18.71	5.49	1.48	0.96
*A 9	5 x5	⅞	27.2	7.99	1.57	17.75	5.17	1.49	0.96
*A 10	5 x5	13/16	25.4	7.46	1.55	16.77	4.85	1.50	0.97
*A 11	5 x5	¾	23.6	6.94	1.52	15.74	4.53	1.51	0.97
*A 12	5 x5	11/16	21.8	6.42	1.50	14.68	4.20	1.51	0.97
*A 13	5 x5	⅝	20.0	5.86	1.48	13.58	3.86	1.52	0.97
*A 14	5 x5	9/16	18.1	5.31	1.46	12.44	3.51	1.53	0.98
*A 15	5 x5	½	16.2	4.75	1.43	11.25	3.15	1.54	0.98
*A 16	5 x5	7/16	14.3	4.18	1.41	10.02	2.79	1.55	0.98
*A 17	5 x5	⅜	12.3	3.61	1.39	8.74	2.42	1.56	0.99
A 18	4 x4	13/16	19.9	5.84	1.29	8.14	3.01	1.18	0.77
A 19	4 x4	¾	18.5	5.41	1.27	7.67	2.81	1.19	0.77

1	2	3	4	5	6	7	8	9	10
Section Index	Size Inches	Thickness, Inches	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity from Back of Flange, Inches	Moment of Inertia, Neutral Axis through Center of Gravity Parallel to Flange I	Section Modulus Neutral Axis as before S	Radius of Gyration Neutral Axis as before r	Least Radius of Gyration, Neutral Axis through Center of Gravity at Angle of 45 Degrees to Flanges r
A 20	4 x4	11/16	17.1	5.03	1.25	7.17	2.61	1.19	0.77
A 21	4 x4	⅝	15.7	4.61	1.23	6.66	2.40	1.20	0.77
A 22	4 x4	9/16	14.3	4.18	1.21	6.12	2.19	1.21	0.78
A 23	4 x4	½	12.8	3.75	1.18	5.56	1.97	1.22	0.78
A 24	4 x4	7/16	11.3	3.31	1.16	4.97	1.75	1.23	0.78
A 25	4 x4	⅜	9.8	2.86	1.14	4.36	1.52	1.23	0.79
A 90	4 x4	5/16	8.2	2.40	1.12	3.71	1.29	1.24	0.79
A 26	3½x3½	13/16	17.1	5.03	1.17	5.25	2.25	1.02	0.67
A 27	3½x3½	¾	16.0	4.69	1.15	4.96	2.11	1.03	0.67
A 28	3½x3½	11/16	14.8	4.34	1.12	4.65	1.96	1.04	0.67
A 29	3½x3½	⅝	13.6	3.98	1.10	4.33	1.81	1.04	0.67
A 30	3½x3½	9/16	12.4	3.62	1.08	3.99	1.65	1.05	0.68
A 31	3½x3½	½	11.1	3.25	1.06	3.64	1.49	1.06	0.68
A 32	3½x3½	7/16	9.8	2.87	1.04	3.26	1.32	1.07	0.68
A 33	3½x3½	⅜	8.5	2.48	1.01	2.87	1.15	1.07	0.69
A 99	3½x3½	5/16	7.2	2.09	0.99	2.45	0.98	1.08	0.69
A 34	3 x3	⅝	11.5	3.36	0.98	2.62	1.30	0.88	0.57
A 35	3 x3	9/16	10.4	3.06	0.95	2.43	1.19	0.89	0.58
A 36	3 x3	½	9.4	2.75	0.93	2.22	1.07	0.90	0.58
A 37	3 x3	7/16	8.3	2.43	0.91	1.99	0.95	0.91	0.58
A 38	3 x3	⅜	7.2	2.11	0.89	1.76	0.83	0.91	0.58
A 39	3 x3	5/16	6.1	1.78	0.87	1.51	0.71	0.92	0.59
A 40	3 x3	¼	4.9	1.44	0.84	1.24	0.58	0.93	0.59
*A 41	2¾x2¾	½	8.5	2.50	0.87	1.67	0.89	0.82	0.52
*A 42	2¾x2¾	7/16	7.6	2.22	0.85	1.51	0.79	0.82	0.53
*A 43	2¾x2¾	⅜	6.6	1.92	0.82	1.33	0.69	0.83	0.53
*A 44	2¾x2¾	5/16	5.6	1.62	0.80	1.13	0.59	0.84	0.54
*A 45	2¾x2¾	¼	4.5	1.31	0.78	0.93	0.48	0.85	0.55
A 46	2½x2½	½	7.7	2.25	0.81	1.23	0.73	0.74	0.47
A 47	2½x2½	7/16	6.8	2.00	0.78	1.11	0.65	0.74	0.48
A 48	2½x2½	⅜	5.9	1.73	0.76	0.98	0.57	0.75	0.48
A 49	2½x2½	5/16	5.0	1.47	0.74	0.85	0.48	0.76	0.49
A 50	2½x2½	¼	4.1	1.19	0.73	0.70	0.40	0.77	0.49
A100	2½x2½	3/16	3.1	0.90	0.69	0.55	0.30	0.78	0.49

Angles marked * are special,

TABLE VII—(Concluded.)

Properties of Standard and Special Angles.

Angles with Equal Legs.

1	2	3	4	5	6	7	8	9	10
Section Index	Size Inches	Thickness, Inches	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity from Back of Flange, Inches	Moment of Inertia, Neutral Axis through Center of Gravity Parallel to Flange I	Section Modulus Neutral Axis as before S	Radius of Gyration Neutral Axis as before r	Least Radius of Gyration, Neutral Axis through Center of Gravity at Angle of 45 Degrees to Flanges r
*A 51	2¼x2¼	½	6.8	2.00	0.74	0.87	0.58	0.66	0.43
*A 52	2¼x2¼	7/16	6.1	1.78	0.72	0.79	0.52	0.67	0.43
*A 53	2¼x2¼	⅜	5.3	1.55	0.70	0.70	0.45	0.67	0.43
*A 54	2¼x2¼	5/16	4.5	1.31	0.68	0.61	0.39	0.68	0.44
*A 55	2¼x2¼	¼	3.7	1.06	0.66	0.51	0.32	0.69	0.44
*A101	2¼x2¼	3/16	2.8	0.81	0.63	0.39	0.24	0.70	0.44
A 56	2 x2	7/16	5.3	1.56	0.66	0.54	0.40	0.59	0.39
A 57	2 x2	⅜	4.7	1.36	0.64	0.48	0.35	0.59	0.39
A 58	2 x2	5/16	4.0	1.15	0.61	0.42	0.30	0.60	0.39
A 59	2 x2	¼	3.2	0.94	0.59	0.35	0.25	0.61	0.39
A 60	2 x2	3/16	2.5	0.72	0.57	0.28	0.19	0.62	0.40
A 61	1¾x1¾	7/16	4.6	1.30	0.59	0.35	0.30	0.51	0.33
A 62	1¾x1¾	⅜	4.0	1.17	0.57	0.31	0.26	0.51	0.34
A 63	1¾x1¾	5/16	3.4	1.00	0.55	0.27	0.23	0.52	0.34
A 64	1¾x1¾	¼	2.8	0.81	0.53	0.23	0.19	0.53	0.34
A 65	1¾x1¾	3/16	2.2	0.62	0.51	0.18	0.14	0.54	0.35
A 66	1½x1½	⅜	3.4	0.99	0.51	0.19	0.19	0.44	0.29
A 67	1½x1½	5/16	2.9	0.84	0.49	0.16	0.162	0.44	0.29
A 68	1½x1½	¼	2.4	0.69	0.47	0.14	0.134	0.45	0.29
A 69	1½x1½	3/16	1.8	0.53	0.44	0.11	0.104	0.46	0.29
A102	1½x1½	⅛	1.3	0.36	0.42	0.08	0.070	0.46	0.30
A 70	1¼x1¼	5/16	2.4	0.69	0.42	0.09	0.109	0.36	0.23
A 71	1¼x1¼	¼	2.0	0.56	0.40	0.077	0.091	0.37	0.24
A 72	1¼x1¼	3/16	1.5	0.43	0.38	0.061	0.071	0.38	0.24
A 73	1¼x1¼	⅛	1.1	0.30	0.35	0.044	0.049	0.38	0.25
A 78	1 x1	¼	1.5	0.44	0.34	0.037	0.056	0.29	0.19
A 79	1 x1	3/16	1.2	0.34	0.32	0.030	0.044	0.30	0.19
A 80	1 x1	⅛	0.8	0.24	0.30	0.022	0.031	0.31	0.20
*A 81	⅞x ⅞	3/16	1.0	0.29	0.29	0.019	0.033	0.26	0.18
*A 82	⅞x ⅞	⅛	0.7	0.21	0.26	0.014	0.023	0.26	0.19
A 83	¾x ¾	3/16	0.9	0.25	0.26	0.012	0.024	0.22	0.16
A 84	¾x ¾	⅛	0.6	0.17	0.23	0.009	0.017	0.23	0.17

Angles marked * are special.

The section modulus about the axis 2-2 is not given in the tables, because the beam is rarely used in this position. It can, however, be readily obtained :

$$S_{2\,2} = \frac{42.538}{12} = 3.55$$

COEFFICIENT OF STRENGTH. This also is a constant employed to express the relations of certain values used in the calculation of stresses in beams. As stated before, $M = \frac{fI}{y}$.

Also $M = \frac{1}{8}\,pl^2$ for a load uniformly distributed, where p = the load per linear foot, and l = the length of span in feet. As the value of M in the first equation is in inch-pounds, in the second also it must be in inch-pounds in order to equate them.

$$\text{Therefore,}\quad M = \frac{fI}{y} = \frac{12\,pl^2}{8},$$

$$\text{and}\ \frac{8M}{12} = pl^2 = C;$$

$$\text{also,}\ \frac{8fI}{12y} = pl^2 = C.$$

This value of C is convenient to use, because from it the total load that a beam can safely carry on a given span is readily obtained.

$$L = \text{total load} = pl = \frac{C}{l}.$$

To derive the value of C in the case of the beam above, if we use $f = 16{,}000$, which is the value for buildings, then

$$C = \frac{8 \times 16{,}000 \times 2{,}087.548}{12 \times 12}$$

$$= 1{,}855{,}598.$$

If the value of I given in the table of Carnegie's Handbook be used, the value of C will check with that above.

The value of C, however, varies as much as or more than this in the different books, because a slight variation in I is multiplied to such an extent. The variation, however, is of no practical importance in deducing the value of L, as the variation here is slight.

C′, the coefficient derived by using the value of $f = 12{,}500$, becomes

$$C' = \frac{8 \times 12{,}500 \times 2{,}087.548}{12 \times 12}$$

$$= 1{,}449{,}685.$$

EQUAL RADII OF GYRATION. The last column in the table is very useful in the designing of members that are desired to be equally strong against bending both in the directon of the web and in a direction at right angles to it, as this column gives the distance apart that beams must be spaced to accomplish this result.

Since the radius of gyration depends on the moment of inertia and the area of section, it follows that for a given section, the radius of gyration about two axes will be proportional to their moments of inertia about these axes.

If d equals the distance in inches from the center of each beam to the neutral axis of the two, in order to obtain $I_{1\text{-}1} = I_{3\text{-}3}$ we must have $I_{1\text{-}1} = I_{2\text{-}2} + A\ d^2$.

In the above case

$$I_{1\text{-}1} - I_{2\text{-}2} = 2{,}087.548 - 42.538 = 23.323\ d^2$$

$$d = \sqrt{\frac{2{,}045.01}{23.323}}$$

$$= 9.35$$

Therefore $\quad D = 2 \times 9.35 = 18.70.$

TABLE VIII.

Properties of Standard and Special Angles.

Angles with Unequal Legs.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Section Index	Size Inches	Thickness Inches	Weight per Foot Pounds	Area of Section Square Inches	Perpendicular Distances from Center of Gravity to Back of Flanges		Moments of Inertia I		Section Moduli S		Radii of Gyration r			Section Index
					To Back of Longer Flange	To Back of Shorter Flange	Neutral Axis Parallel to Longer Flange	Neutral Axis Parallel to Shorter Flange	Neutral Axis Parallel to Longer Flange	Neutral Axis Parallel to Shorter Flange	Neutral Axis Parallel to Longer Flange	Neutral Axis Parallel to Shorter Flange	Least Radius	
*A140	8 x3½	11/20	20.5	6.02	0.75	3.00	4.92	39.96	1.79	7.99	0.90	2.58	0.74	A140*
*A150	7 x3½	1	32.3	9.50	0.96	2.71	7.53	45.37	2.96	10.58	0.89	2.19	0.88	A150*
*A151	7 x3½	15/16	30.5	8.97	0.94	2.69	7.18	43.13	2.80	10.00	0.89	2.19	0.88	A151*
*A152	7 x3½	7/8	28.7	8.42	0.92	2.67	6.83	40.82	2.64	9.42	0.90	2.20	0.88	A152*
*A153	7 x3½	13/16	26.8	7.87	0.89	2.64	6.46	38.45	2.48	8.82	0.91	2.21	0.88	A153*
*A154	7 x3½	3/4	24.9	7.31	0.87	2.62	6.08	35.99	2.31	8.22	0.91	2.22	0.88	A154*
*A155	7 x3½	11/16	23.0	6.75	0.85	2.60	5.69	33.47	2.14	7.60	0.92	2.23	0.89	A155*
*A156	7 x3½	5/8	21.0	6.17	0.82	2.57	5.28	30.86	1.97	6.97	0.93	2.24	0.89	A156*
*A157	7 x3½	9/16	19.1	5.59	0.80	2.55	4.86	28.18	1.80	6.33	0.93	2.25	0.89	A157*
*A158	7 x3½	1/2	17.0	5.00	0.78	2.53	4.41	25.41	1.62	5.68	0.94	2.25	0.89	A158*
*A159	7 x3½	7/16	15.0	4.40	0.75	2.50	3.95	22.56	1.47	5.01	0.95	2.26	0.89	A159*
A 89	6 x4	1	30.6	9.00	1.17	2.17	10.75	30.75	3.79	8.02	1.09	1.85	0.85	A 89
A 91	6 x4	15/16	28.9	8.50	1.14	2.14	10.26	29.26	3.59	7.59	1.10	1.86	0.85	A 91
A160	6 x4	7/8	27.2	7.99	1.12	2.12	9.75	27.73	3.39	7.15	1.11	1.86	0.86	A160
A161	6 x4	13/16	25.4	7.47	1.10	2.10	9.23	26.15	3.18	6.70	1.11	1.87	0.86	A161
A162	6 x4	3/4	23.6	6.94	1.08	2.08	8.68	24.51	2.97	6.25	1.12	1.88	0.86	A162
A163	6 x4	11/16	21.8	6.41	1.06	2.06	8.11	22.82	2.76	5.78	1.13	1.89	0.86	A163
A164	6 x4	5/8	20.0	5.86	1.03	2.03	7.52	21.07	2.54	5.31	1.13	1.90	0.86	A164
A165	6 x4	9/16	18.1	5.31	1.01	2.01	6.91	19.26	2.31	4.83	1.14	1.90	0.87	A165
A166	6 x4	1/2	16.2	4.75	0.99	1.99	6.27	17.40	2.08	4.33	1.15	1.91	0.87	A166
A167	6 x4	7/16	14.3	4.18	0.96	1.96	5.60	15.46	1.85	3.83	1.16	1.92	0.87	A167
A168	6 x4	3/8	12.3	3.61	0.94	1.94	4.90	13.47	1.60	3.32	1.17	1.93	0.88	A168
A 92	6 x3½	1	28.9	8.50	1.01	2.26	7.21	29.24	2.90	7.83	0.92	1.85	0.74	A 92
A 93	6 x3½	15/16	27.3	8.03	0.99	2.24	6.88	27.84	2.71	7.41	0.93	1.86	0.74	A 93
A169	6 x3½	7/8	25.7	7.55	0.97	2.22	6.55	26.38	2.59	6.98	0.93	1.87	0.75	A169
A170	6 x3½	13/16	24.0	7.06	0.95	2.20	6.20	24.89	2.43	6.55	0.94	1.88	0.75	A170
A171	6 x3½	3/4	22.4	6.56	0.93	2.18	5.84	23.34	2.27	6.10	0.94	1.89	0.75	A171
A172	6 x3½	11/16	20.6	6.06	0.90	2.15	5.47	21.74	2.11	5.65	0.95	1.89	0.75	A172
A173	6 x3½	5/8	18.9	5.55	0.88	2.13	5.08	20.08	1.94	5.19	0.96	1.90	0.75	A173
A174	6 x3½	9/16	17.1	5.03	0.86	2.11	4.67	18.37	1.77	4.72	0.96	1.91	0.75	A174
A175	6 x3½	1/2	15.3	4.50	0.83	2.08	4.25	16.59	1.59	4.24	0.97	1.92	0.76	A175
A176	6 x3½	7/16	13.5	3.97	0.81	2.06	3.81	14.76	1.41	3.75	0.98	1.93	0.76	A176
A177	6 x3½	3/8	11.7	3.42	0.79	2.04	3.34	12.86	1.23	3.25	0.99	1.94	0.77	A177
*A178	5 x4	7/8	24.2	7.11	1.21	1.71	9.23	16.42	3.31	4.99	1.14	1.52	0.84	A178*
*A179	5 x4	13/16	22.7	6.65	1.18	1.68	8.74	15.54	3.11	4.69	1.15	1.53	0.84	A179*
*A180	5 x4	3/4	21.1	6.19	1.16	1.66	8.23	14.60	2.90	4.37	1.15	1.54	0.84	A180*

Angles marked * are special.

TABLE VIII — (Continued.)

Properties of Standard and Special Angles.

Angles with Unequal Legs.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Section Index	Size Inches	Thickness Inches	Weight per Foot Pounds	Area of Section Square Inches	Perpendicular Distances from Center of Gravity to Back of Flanges		Moments of Inertia I		Section Moduli S		Radii of Gyration r			Section Index
					To Back of Longer Flange	To Back of Shorter Flange	Neutral Axis Parallel to Longer Flange	Neutral Axis Parallel to Shorter Flange	Neutral Axis Parallel to Longer Flange	Neutral Axis Parallel to Shorter Flange	Neutral Axis Parallel to Longer Flange	Neutral Axis Parallel to Shorter Flange	Least Radius	
*A181	5 x4	11/16	19.5	5.72	1.14	1.64	7.70	13.62	2.69	4.05	1.16	1.54	0.84	A181*
*A182	5 x4	5/8	17.8	5.23	1.12	1.62	7.14	12.61	2.48	3.73	1.17	1.55	0.84	A182*
*A183	5 x4	9/16	16.2	4.75	1.10	1.60	6.56	11.55	2.26	3.39	1.18	1.56	0.85	A183*
*A184	5 x4	1/2	14.5	4.25	1.07	1.57	5.96	10.46	2.04	3.05	1.18	1.57	0.85	A184*
*A185	5 x4	7/16	12.8	3.75	1.05	1.55	5.32	9.32	1.81	2.70	1.19	1.58	0.85	A185*
*A186	5 x4	3/8	11.0	3.23	1.03	1.53	4.67	8.14	1.57	2.34	1.20	1.59	0.86	A186*
A187	5 x3½	7/8	22.7	6.67	1.04	1.79	6.21	15.67	2.52	4.88	0.96	1.53	0.75	A187
A188	5 x3½	13/16	21.3	6.25	1.02	1.77	5.89	14.81	2.37	4.58	0.97	1.54	0.75	A188
A189	5 x3½	3/4	19.8	5.81	1.00	1.75	5.55	13.92	2.22	4.23	0.98	1.55	0.75	A189
A190	5 x3½	11/16	18.3	5.37	0.97	1.72	5.20	12.99	2.06	3.97	0.98	1.56	0.75	A190
A191	5 x3½	5/8	16.8	4.92	0.95	1.70	4.83	12.03	1.90	3.65	0.99	1.56	0.75	A191
A192	5 x3½	9/16	15.2	4.47	0.93	1.68	4.45	11.03	1.73	3.32	1.00	1.57	0.75	A192
A193	5 x3½	1/2	13.6	4.00	0.91	1.66	4.05	9.99	1.56	2.99	1.01	1.58	0.75	A193
A194	5 x3½	7/16	12.0	3.53	0.88	1.63	3.63	8.90	1.39	2.64	1.01	1.59	0.76	A194
A195	5 x3½	3/8	10.4	3.05	0.86	1.61	3.18	7.78	1.21	2.29	1.02	1.60	0.76	A195
A 96	5 x3½	5/16	8.7	2.56	0.84	1.59	2.72	6.60	1.02	1.94	1.03	1.61	0.76	A 96
A196	5 x3	13/16	19.9	5.84	0.86	1.86	3.71	13.98	1.74	4.45	0.80	1.55	0.64	A196
A197	5 x3	3/4	18.5	5.44	0.84	1.84	3.51	13.15	1.63	4.16	0.80	1.55	0.64	A197
A198	5 x3	11/16	17.1	5.03	0.82	1.82	3.29	12.28	1.51	3.86	0.81	1.56	0.64	A198
A199	5 x3	5/8	15.7	4.61	0.80	1.80	3.06	11.37	1.39	3.55	0.82	1.57	0.64	A199
A200	5 x3	9/16	14.3	4.18	0.77	1.77	2.83	10.43	1.27	3.23	0.82	1.58	0.65	A200
A201	5 x3	1/2	12.8	3.75	0.75	1.75	2.58	9.45	1.15	2.91	0.83	1.59	0.65	A201
A202	5 x3	7/16	11.3	3.31	0.73	1.73	2.32	8.43	1.02	2.58	0.84	1.60	0.65	A203
A203	5 x3	3/8	9.8	2.86	0.70	1.70	2.04	7.37	0.89	2.24	0.84	1.61	0.65	A203
A280	5 x3	5/16	8.2	2.40	0.68	1.68	1.75	6.26	0.75	1.89	0.85	1.61	0.66	A280
*A204	4½x3	13/16	18.5	5.43	0.90	1.65	3.60	10.33	1.71	3.62	0.81	1.38	0.64	A204*
*A205	4½x3	3/4	17.3	5.06	0.88	1.63	3.40	9.73	1.60	3.38	0.82	1.39	0.64	A205*
*A206	4½x3	11/16	16.0	4.68	0.85	1.60	3.19	9.10	1.49	3.14	0.83	1.39	0.64	A206*
*A207	4½x3	5/8	14.7	4.30	0.83	1.58	2.98	8.44	1.37	2.89	0.83	1.40	0.64	A207*
*A208	4½x3	9/16	13.3	3.90	0.81	1.56	2.75	7.75	1.25	2.61	0.85	1.41	0.64	A208*
*A209	4½x3	1/2	11.9	3.50	0.79	1.54	2.51	7.04	1.13	2.37	0.85	1.42	0.65	A209*
*A210	4½x3	7/16	10.6	3.09	0.76	1.51	2.25	6.29	1.01	2.10	0.85	1.43	0.65	A210*
*A211	4½x3	3/8	9.1	2.67	0.74	1.49	1.98	5.50	0.88	1.83	0.86	1.44	0.66	A211*
*A 97	4½x3	5/16	7.7	2.25	0.72	1.47	1.73	4.69	0.76	1.54	0.88	1.44	0.66	A 97*

Angles marked * are special.

TABLE VIII —(Continued.)

Properties of Standard and Special Angles.

Angles with Unequal Legs.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
					Perpendicular Distances from Center of Gravity to Back of Flanges		Moments of Inertia I		Section Moduli S		Radii of Gyration r			
Section Index	Size Inches	Thickness Inches	Weight per Foot Pounds	Area of Section Square Inches	To Back of Longer Flange	To Back of Shorter Flange	Neutral Axis Parallel to Longer Flange	Neutral Axis Parallel to Shorter Flange	Neutral Axis Parallel to Longer Flange	Neutral Axis Parallel to Shorter Flange	Neutral Axis Parallel to Longer Flange	Neutral Axis Parallel to Shorter Flange	Least Radius	Section Index
*A212	4 x3½	13/16	18.5	5.43	1.11	1.36	5.49	7.77	2.30	2.92	1.01	1.19	0.72	A212*
*A213	4 x3½	3/4	17.3	5.05	1.09	1.34	5.18	7.32	2.15	2.75	1.01	1.20	0.72	A213*
*A214	4 x3½	11/16	16.0	4.68	1.07	1.32	4.86	6.86	2.00	2.56	1.02	1.21	0.72	A214*
*A215	4 x3½	5/8	14.7	4.30	1.04	1.29	4.52	6.37	1.84	2.35	1.03	1.22	0.72	A215*
*A216	4 x3½	9/16	13.3	3.90	1.03	1.27	4.17	5.86	1.68	2.15	1.03	1.23	0.72	A216*
*A217	4 x3½	1/2	11.9	3.50	1.00	1.25	3.79	5.32	1.52	1.93	1.04	1.23	0.72	A217*
*A218	4 x3½	7/16	10.6	3.09	0.98	1.23	3.40	4.76	1.35	1.72	1.05	1.24	0.72	A218*
*A219	4 x3½	3/8	9.1	2.67	0.96	1.21	2.99	4.18	1.18	1.50	1.06	1.25	0.73	A219*
*A 98	4 x3½	5/16	7.7	2.25	0.93	1.18	2.59	3.56	1.01	1.26	1.07	1.26	0.73	A 98*
A220	4 x3	13/16	17.1	5.03	0.94	1.44	3.47	7.24	1.68	2.87	0.83	1.21	0.64	A220
A221	4 x3	3/4	16.0	4.69	0.92	1.42	3.28	6.93	1.57	2.68	0.84	1.22	0.64	A221
A222	4 x3	11/16	14.8	4.34	0.89	1.39	3.08	6.49	1.46	2.49	0.84	1.22	0.64	A222
A223	4 x3	5/8	13.6	3.98	0.87	1.37	2.87	6.03	1.35	2.30	0.85	1.23	0.64	A223
A224	4 x3	9/16	12.4	3.62	0.85	1.35	2.66	5.55	1.23	2.09	0.86	1.24	0.64	A224
A225	4 x3	1/2	11.1	3.25	0.83	1.33	2.42	5.05	1.12	1.89	0.86	1.25	0.64	A225
A226	4 x3	7/16	9.8	2.87	0.80	1.30	2.18	4.52	0.99	1.68	0.87	1.25	0.64	A226
A227	4 x3	3/8	8.5	2.48	0.78	1.28	1.92	3.96	0.87	1.46	0.88	1.26	0.64	A227
A228	4 x3	5/16	7.2	2.09	0.76	1.26	1.65	3.33	0.71	1.23	0.89	1.27	0.65	A228
A229	3½x3	13/16	15.8	4.03	0.98	1.23	3.33	4.93	1.65	2.20	0.85	1.04	0.62	A229
A230	3½x3	3/4	14.7	4.31	0.96	1.21	3.15	4.70	1.54	2.05	0.85	1.04	0.62	A230
A231	3½x3	11/16	13.6	4.00	0.94	1.19	2.96	4.41	1.44	1.91	0.86	1.05	0.62	A231
A232	3½x3	5/8	12.5	3.67	0.92	1.17	2.76	4.11	1.33	1.76	0.87	1.06	0.62	A232
A233	3½x3	9/16	11.4	3.34	0.90	1.15	2.55	3.79	1.21	1.61	0.87	1.07	0.62	A233
A234	3½x3	1/2	10.2	3.00	0.88	1.13	2.33	3.45	1.10	1.45	0.88	1.07	0.62	A234
A235	3½x3	7/16	9.1	2.65	0.85	1.10	2.09	3.10	0.98	1.29	0.89	1.08	0.62	A235
A236	3½x3	3/8	7.9	2.30	0.83	1.08	1.85	2.72	0.85	1.13	0.90	1.09	0.62	A236
A237	3½x3	5/16	6.6	1.93	0.81	1.06	1.58	2.33	0.72	0.96	0.90	1.10	0.63	A237
A238	3½x2½	11/16	12.5	3.65	0.77	1.27	1.72	4.13	0.99	1.85	0.67	1.06	0.53	A238
A239	3½x2½	5/8	11.5	3.36	0.75	1.25	1.61	3.85	0.92	1.71	0.69	1.07	0.53	A239
A240	3½x2½	9/16	10.4	3.05	0.73	1.23	1.49	3.55	0.84	1.56	0.70	1.08	0.53	A240
A241	3½x2½	1/2	9.4	2.75	0.70	1.20	1.36	3.24	0.76	1.41	0.70	1.09	0.53	A241
A242	3½x2½	7/16	8.3	2.43	0.68	1.18	1.23	2.91	0.63	1.26	0.71	1.09	0.54	A242
A243	3½x2½	3/8	7.2	2.11	0.66	1.16	1.09	2.56	0.59	1.09	0.72	1.10	0.54	A243
A244	3½x2½	5/16	6.1	1.73	0.64	1.14	0.94	2.19	0.50	0.93	0.73	1.11	0.54	A244
A245	3½x2½	1/4	4.9	1.44	0.61	1.11	0.78	1.80	0.41	0.75	0.74	1.12	0.54	A245

Angles marked * are special.

TABLE VIII—(Concluded.)

Properties of Standard and Special Angles.

Angles with Unequal Legs.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Section Index	Size Inches	Thickness Inches	Weight per Foot Pounds	Area of Section Square Inches	Perpendicular Distances from Center of Gravity to Back of Flanges		Moments of Inertia I		Section Moduli S		Radii of Gyration r			Section Index
					To Back of Longer Flange	To Back of Shorter Flange	Neutral Axis Parallel to Longer Flange	Neutral Axis Parallel to Shorter Flange	Neutral Axis Parallel to Longer Flange	Neutral Axis Parallel to Shorter Flange	Neutral Axis Parallel to Longer Flange	Neutral Axis Parallel to Shorter Flange	Least Radius	
*A246	3¼x2	9/16	9.0	2.64	0.59	1.21	0.75	2.64	0.53	1.30	0.53	1.00	0.44	A246*
*A247	3¼x2	½	8.1	2.38	0.57	1.19	0.69	2.42	0.48	1.17	0.54	1.01	0.44	A247*
*A248	3¼x2	7/16	7.2	2.11	0.54	1.17	0.62	2.18	0.43	1.05	0.54	1.02	0.44	A248*
*A249	3¼x2	⅜	6.3	1.83	0.52	1.15	0.55	1.92	0.37	0.91	0.55	1.02	0.44	A249*
*A250	3¼x2	5/16	5.3	1.54	0.50	1.12	0.48	1.65	0.32	0.77	0.56	1.03	0.45	A250*
*A251	3¼x2	¼	4.3	1.25	0.48	1.09	0.40	1.36	0.26	0.63	0.57	1.04	0.45	A251*
A252	3 x2½	9/16	9.5	2.78	0.77	1.02	1.42	2.28	0.82	1.15	0.72	0.91	0.52	A252
A253	3 x2½	½	8.5	2.50	0.75	1.00	1.30	2.08	0.74	1.04	0.72	0.91	0.52	A253
A254	3 x2½	7/16	7.6	2.22	0.73	0.98	1.18	1.88	0.66	0.93	0.73	0.92	0.52	A254
A255	3 x2½	⅜	6.6	1.92	0.71	0.96	1.04	1.66	0.58	0.81	0.74	0.93	0.52	A255
A256	3 x2½	5/16	5.6	1.62	0.68	0.93	0.90	1.42	0.49	0.69	0.74	0.94	0.53	A256
A257	3 x2½	¼	4.5	1.31	0.66	0.91	0.74	1.17	0.40	0.56	0.75	0.95	0.53	A257
*A258	3 x2	½	7.7	2.25	0.58	1.08	0.67	1.92	0.47	1.00	0.55	0.92	0.43	A258*
*A259	3 x2	7/16	6.8	2.00	0.56	1.06	0.61	1.73	0.42	0.89	0.55	0.93	0.43	A259*
*A260	3 x2	⅜	5.9	1.73	0.54	1.04	0.54	1.53	0.37	0.78	0.56	0.94	0.43	A260*
*A261	3 x2	5/16	5.0	1.47	0.52	1.02	0.47	1.32	0.32	0.66	0.57	0.95	0.43	A261*
*A262	3 x2	¼	4.1	1.19	0.49	0.99	0.39	1.09	0.25	0.54	0.57	0.95	0.43	A262*
A264	2½x2	½	6.8	2.00	0.63	0.88	0.64	1.14	0.46	0.70	0.56	0.75	0.42	A264
A265	2½x2	7/16	6.1	1.78	0.60	0.85	0.58	1.03	0.41	0.62	0.57	0.76	0.42	A265
A266	2½x2	⅜	5.3	1.55	0.58	0.83	0.51	0.91	0.36	0.55	0.58	0.77	0.42	A266
A267	2½x2	5/16	4.5	1.31	0.56	0.81	0.45	0.79	0.31	0.47	0.58	0.78	0.42	A267
A268	2½x2	¼	3.7	1.06	0.54	0.79	0.37	0.65	0.25	0.38	0.59	0.78	0.42	A268
A269	2½x2	3/16	2.8	0.81	0.51	0.76	0.29	0.51	0.20	0.29	0.60	0.79	0.43	A269
*A270	2¼x1½	½	5.6	1.63	0.48	0.86	0.26	0.75	0.26	0.54	0.40	0.68	0.39	A270*
*A271	2¼x1½	7/16	5.0	1.45	0.46	0.83	0.24	0.68	0.23	0.48	0.41	0.69	0.39	A271*
*A272	2¼x1½	⅜	4.4	1.27	0.44	0.81	0.21	0.61	0.20	0.42	0.41	0.69	0.39	A272*
*A273	2¼x1½	5/16	3.7	1.07	0.42	0.79	0.19	0.53	0.17	0.36	0.42	0.70	0.40	A273*
*A274	2¼x1½	¼	3.0	0.88	0.39	0.77	0.16	0.44	0.14	0.30	0.42	0.71	0.40	A274*
*A275	2¼x1½	3/16	2.3	0.67	0.37	0.75	0.12	0.34	0.11	0.23	0.43	0.72	0.40	A275*
*A276	2 x1⅜	¼	2.7	0.78	0.37	0.69	0.12	0.37	0.12	0.23	0.39	0.63	0.30	A276*
*A277	2 x1⅜	3/16	2.1	0.60	0.35	0.66	0.09	0.24	0.09	0.18	0.40	0.63	0.31	A277*
*A278	1⅜x1	¼	1.9	0.53	0.29	0.48	0.04	0.09	0.05	0.09	0.27	0.41	0.22	A278*
*A279	1⅜x1	⅛	1.0	0.28	0.26	0.44	0.02	0.05	0.03	0.06	0.29	0.44	0.22	A279*

Angles marked * are special.

BUILDING LAWS AND SPECIFICATIONS.

The requirements of the Building Departments of different cities vary considerably as regards detail matters, but are in quite close agreement on points affecting the strength of structures.

The following table shows the requirements of different cities as regards live loads:

TABLE IX.

Building Laws: — Specified Live Loads in Different Classes of Buildings.

The loads specified are exclusive of weight of materials of construction.

Class of Structure.	New York, 1900.	Chicago, 1900.	Philadelphia, 1903.	Boston, 1900.
	Load, Pounds per Square Foot.			
Dwellings, Apartment Houses, Hotels, and Lodging Houses	60	40	70	50
Office Buildings — First Floor	150	100	100	100
Office Buildings — above First Floor	75	100	100	100
Schools — except Assembly Halls	75			80
Assembly Halls	90	100	120	150
*Stores for Heavy Materials; Warehouses and Factories; Drill Sheds . . .	150	100	150	250
Roofs	50	25	30	25

*Minimum loads as above.

Buildings used for special purposes to have loads specified accordingly.

Table X indicates allowable unit-stresses.

TABLE X.

Allowable Unit-stresses for Steel and Cast Iron, as Specified by Building Laws of Different Cities.

Stresses are for Medium Steel unless otherwise noted.

	NEW YORK, 1900.	CHICAGO, 1900.	PHILADELPHIA, 1903.	BOSTON, 1900.
	Pounds per Square Inch.			
Extreme fibre stress-Bending				
Rolled steel beams and shapes	16,000	16,000	16,000	16,000
Rolled steel pins, rivets	20,000	22,500		22,500
Riveted steel beams — Compression				12,000
Riveted steel beams — Tension net section	14,000			15,000
Cast iron — Compression	16,000	10,000		8,000
Cast iron — Tension	3,000	2,500	3,750	2,500
Compression, Direct.				
Rolled steel	16,000		16,250	
Cast steel	16,000		16,250	
Wrought iron	12,000		12,500	
Cast iron (in short blocks)	16,000		17,500	
Steel pins and rivets (bearing)	20,000	20,000		18,000
Wrought-iron pins and rivets (bearing)	15,000	15,000		15,000
Tension, Direct.				
Rolled steel	16,000	15,000	16,250	15,000
Cast steel	16,000	15,000	16.250	
Wrought iron	12,000	12,000	12,500	12,000
Cast iron	3,000			
Shear.				
Steel web plates	9,000			10,000
Steel shop rivets and pins	10,000	10,000	10,000	10,000
Steel field rivets and pins	8,000	10,000	10,000	10,000
Steel field bolts	7,000		10,000	10,000
Wrought-iron web plate	6,000			9,000
Wrought-iron shop rivets and pins	7,500	7,500	7,500	9,000
Wrought-iron field rivets and pins	6,000	7,500	7,500	9,000
Wrought-iron field bolts	5,500		7,500	9,000
Cast iron	3,000			
Columns.				
Mild steel*	$15\,200-58\frac{L}{R}$	15,000	$\frac{14,500}{1+\frac{L^2}{13,500\ R^2}}$	12,000
Medium steel*	$15,200-58\frac{L}{R}$	15,000	$\frac{16,250}{1+\frac{L^2}{11,000\ R^2}}$	12,000
Wrought iron	$14,000-80\frac{L}{R}$	12,000	$\frac{12,500}{1+\frac{L^2}{15,000\ R^2}}$	10,000
Cast iron	$11,300-30\frac{L}{R}$	10,000	$\frac{17.500}{1+\frac{L^2}{400\ R^2}}$	

* Reduced by Gordon's or other approved formulæ for varying ratios of length to radius

Table XI gives, in pounds per square inch, the transverse strength of various stone constructions, brick and concrete:

TABLE XI.

Transverse Strength of Stone, Brick and Concrete.

Extreme fibre stress — Bending.	POUNDS PER SQUARE INCH.
Blue stone flagging	2,200
Granite	1,700
Limestone	900
Marble	2,000
Slate	5,800
Sandstone	810
Brick	725
Concrete, 1 Port. cem., 2 sand, 5 gravel	200
Concrete, 1 Port. cem., 3 sand, 7 gravel	115

Where walls are carried by the steel framing at each story, they are generally made 12 inches thick.

The question of height also affects the requirements of fire resistance and prevention. There is considerable variation on these points. Table XII gives the requirements of some cities whose laws are explicit as to the thickness of walls and the proportion of loads on columns and foundations.

TABLE XII.

Thickness in Inches of Brick Bearing Walls. Chicago Law, 1901.

STORIES	BASEMENT	1st	2d	3d	4th	5th	6th	7th	8th	9th	10th	11th	12th
One-story	12	12											
Two-story	16	12	12										
Three-story	16	16	12	12									
Four-story	20	20	16	16	12								
Five-story	24	20	20	16	16	16							
Six-story	24	20	20	20	16	16	16						
Seven-story	24	20	20	20	20	16	16	16					
Eight-story	24	24	24	20	20	20	16	16	16				
Nine-story	28	24	24	24	20	20	20	16	16	16			
Ten-story	28	28	28	24	24	24	20	20	20	16	16		
Eleven-story	28	28	28	24	24	24	20	20	20	16	16	16	
Twelve-story	32	28	28	28	24	24	24	20	20	20	16	16	16

The above table applies to manufacturing and storage buildings.

TABLE XII, A.

Thickness in Inches of Brick Bearing Walls. Philadelphia Law, 1903.

STORIES	1st	2d	3d	4th	5th	6th	7th	8th	9th	10th	11th	12th
One-story	13											
Two-story	13	13										
Three-story	18	13	13									
Four-story	18	18	13	13								
Five-story	22	18	18	13	13							
Six-story	22	22	18	18	13	13						
Seven-story	26	22	22	18	18	13	13					
Eight-story	26	26	22	22	18	18	13	13				
Nine-story	30	26	26	22	22	18	18	13	13			
Ten-story	30	30	26	26	22	22	18	18	13	13		
Eleven-story	34	30	30	26	26	22	22	18	18	13	13	
Twelve-story	34	34	30	30	26	26	22	22	18	18	13	13

The above applies to exterior and bearing walls of business, manufacturing and public buildings, 75 feet to 125 feet long and 26 feet or less clear span. Hotels and tenements may have the 3 upper stories 13 inches and following down from that in the sequence given above.

TABLE XII, B.

Safe Bearing Values in Tons per sq. ft. on Different Classes of Masonry.	New York, 1900.	Chicago, 1901.	Philadelphia, 1903.	Boston, 1900.
Granite (Dressed Joints) in Portland Cement	—	7	—	60
Rubble Stonework in Lime Mortar	5	—	5	—
Rubble Stonework in Cement Mortar	10	—	10	—
Brickwork in Lime Mortar	8	6½	8	8
Brickwork in Cement Mortar	15	9	15	15
Concrete, Portland Cement	16	4	15	—
Hardwood Piles (Maximum on Head of Pile)	—	25	20	—

The minimum thickness of curtain walls in Chicago is 12 inches, in Philadelphia 13 inches, and in New York 12 inches for

the upper 75 feet of wall, and 4 inches thicker for each 60 feet below. The New York law allows curtain walls to be built between piers or steel columns and not supported on steel girders, provided the thickness is 12 inches for the upper 60 feet and 4 inches thicker for each 60 feet below.

Wind Pressure. The Philadelphia law requires 30 pounds per square foot to be calculated on exposed surfaces of isolated buildings ; on office buildings 25 pounds per square foot at the 10th floors, and $2\frac{1}{2}$ pounds less for each story below and $2\frac{1}{2}$ pounds more for each story above, up to a maximum of 35 pounds.

The combined stress in columns resulting from direct vertical loads and the bending due to the above wind pressures is allowed to be 30 per cent above that for simply direct loading by the Philadelphia law, and 50 per cent by the New York law.

In New York no allowance for wind is required if the building is under 150 feet high, and this height does not exceed four times the average width of base. For buildings other than as above, 30 pounds per square foot of wind pressure from the ground to the top is required. The overturning moment of the wind is not allowed to be more than 75 per cent of the moment of stability of the structure.

Reduction in Live Load on Columns, Girders and Foundations. The Philadelphia law allows the live loads used in calculation of columns, girders and foundations for all but manufacturing and storage buildings, to be reduced by the following formula:

$$x = 100 - \frac{4}{5} A ;$$

and for light manufacturing buildings, by

$$x = 100 - \frac{2}{5} A,$$

where $x =$ the percentage of live load to be used, and A = the area supported.

The New York law requires the full live load of roof and top floor, but allows a reduction in each succeeding lower floor of 5 per cent until this reduction amounts to 50 per cent of the live load; not less than 50 per cent of the live load may be used in the calculations. For foundations not less than 60 per cent of the live load may be used.

Where the laws limit the height to about 125 feet, the requirements as regards fire protection and prevention are in general that the floors and roofs shall be constructed of steel beams and girders, between which shall be sprung arches of tile or terra cotta or brick, or approved systems of concrete and concrete-steel. All weight-bearing metal of every description shall be covered with non-combustible materials, generally terra cotta or wire lath and cement.

In buildings of this height the use of wood for top floors laid in wood screeds imbedded in concrete, and of wood for all interior finish, is allowed.

Under the New York City law, buildings above sixteen stories are required to have their upper stories constructed entirely without wood, except that the so-called fireproof wood may be used for interior finish. The floors, however, are required to be of tile or mosaic or other non-combustible material, the wood top floor not being allowed.

Factor of Safety. The foregoing values represent the working values of unit-stresses. They are in all cases a certain percentage of the strains under which rupture would occur. This percentage varies with the different classes of material and the different classes of structure. The quotient of the breaking strain divided by the allowable or safe working strain is called the "factor of safety."

Steel and wrought iron used in ordinary building construction have generally a factor of safety of 4; timber, generally from 6 to 8; cast iron, from 6 to 10; stone from 10 to 15.

One reason for this variation in factors of safety for different materials is that certain materials vary more than others in their internal structure; and accordingly in some cases there is a greater likelihood than in others, of an individual piece being below the average strength. Other reasons are found in the varying effects of time. Changes in internal structure are likely to occur in the lapse of years; and there is the further liability that through ignorance or carelessness the structure may be put to uses for which it was never designed.

All these conditions make it unwise from the standpoint of safety to use working stresses very near the breaking strains.

Steel is less subject to variation than other materials. Timber has knots, shakes, dry rot, and other defects not readily discerned, which may greatly reduce its strength below the average. Cast iron has blow-holes, cracks, flaws, internal strains, and unequally distributed metal, which are of frequent occurrence and very

Basement Plan

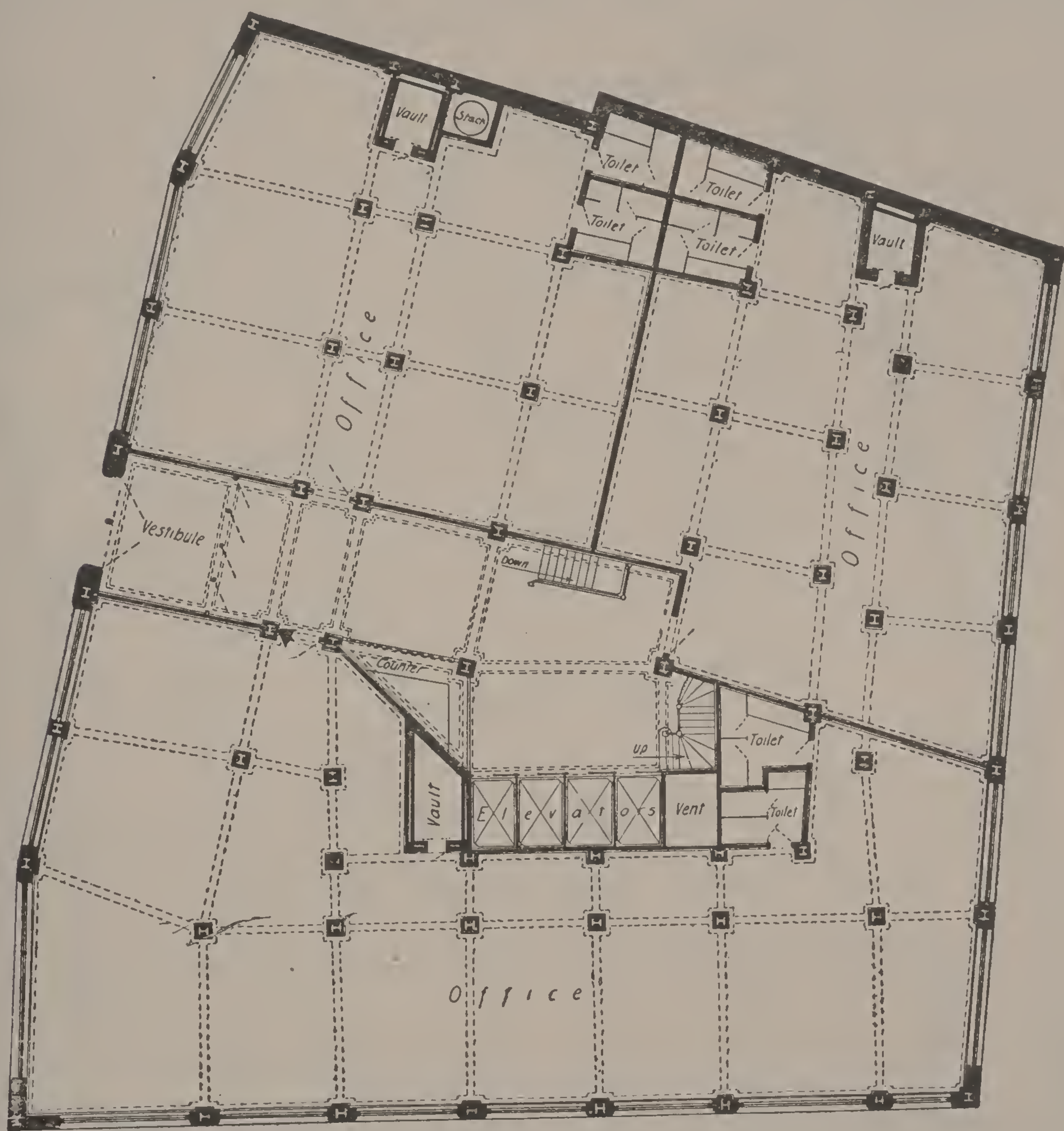
Fig 40

likely to escape detection. Stone has seams, crack , flaws, and a structure not uniform, all causing uncertainty and variations in the strength of individual pieces.

THE STEEL FRAME.

The problems to be met with in laying out the steel frame and designing the different elements are never twice the same but,

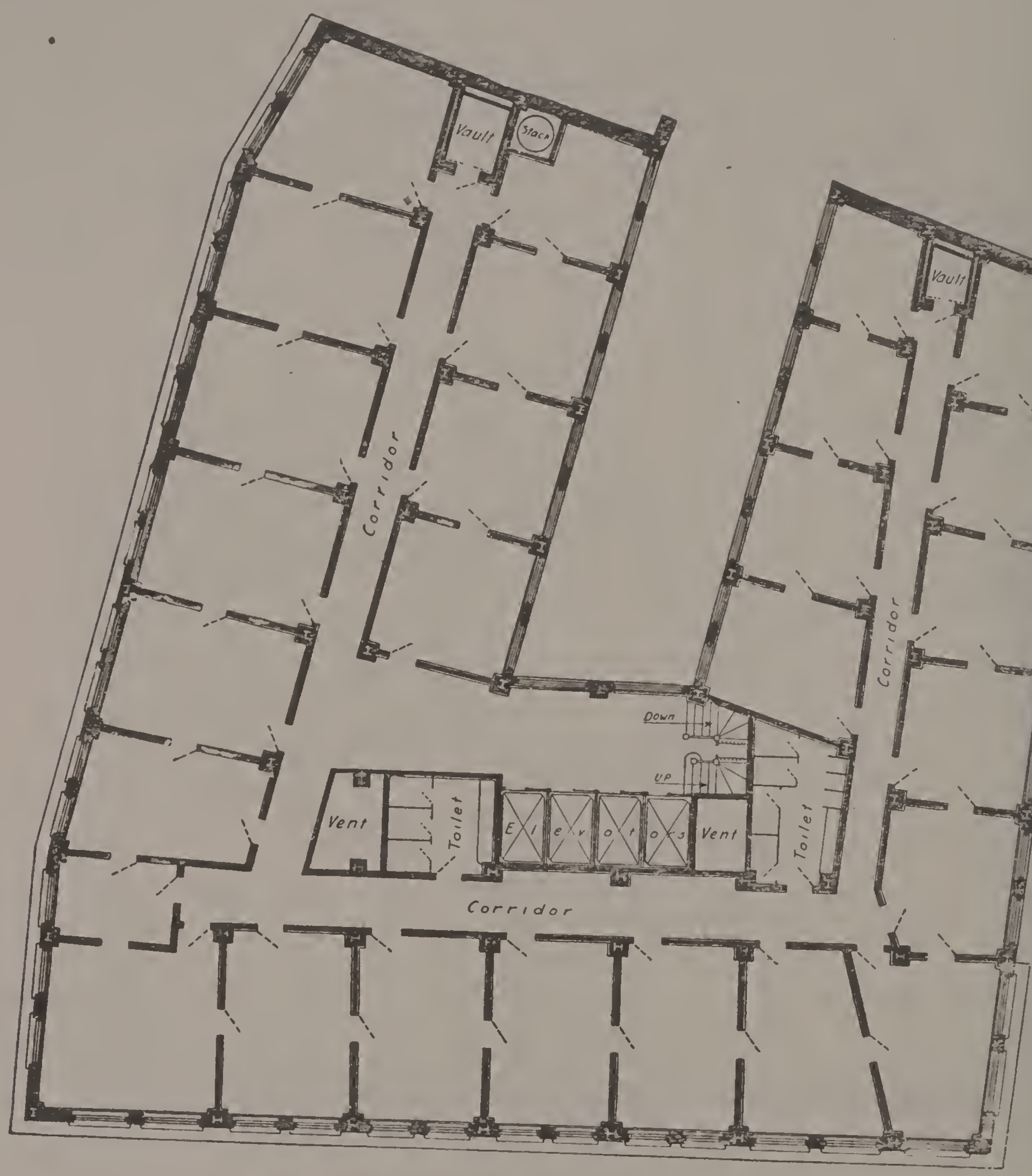
vary with each special case. Different classes of buildings give rise to different problems. Some of the problems that naturally arise can best be explained by going in detail through the process of framing the office building of which plans are given in Figs. 40 to 45.



First Floor Plan
Fig. 41.

In a building of this character, and in all buildings where the interior arrangement is a feature, the designer of the steel frame must base his work on the architect's layout. For this purpose it is most convenient, in making the preliminary study and provisional framing plans, to use tracing paper, which can be placed over the architect's plans, and thus show the position of all partitions, ducts, etc.

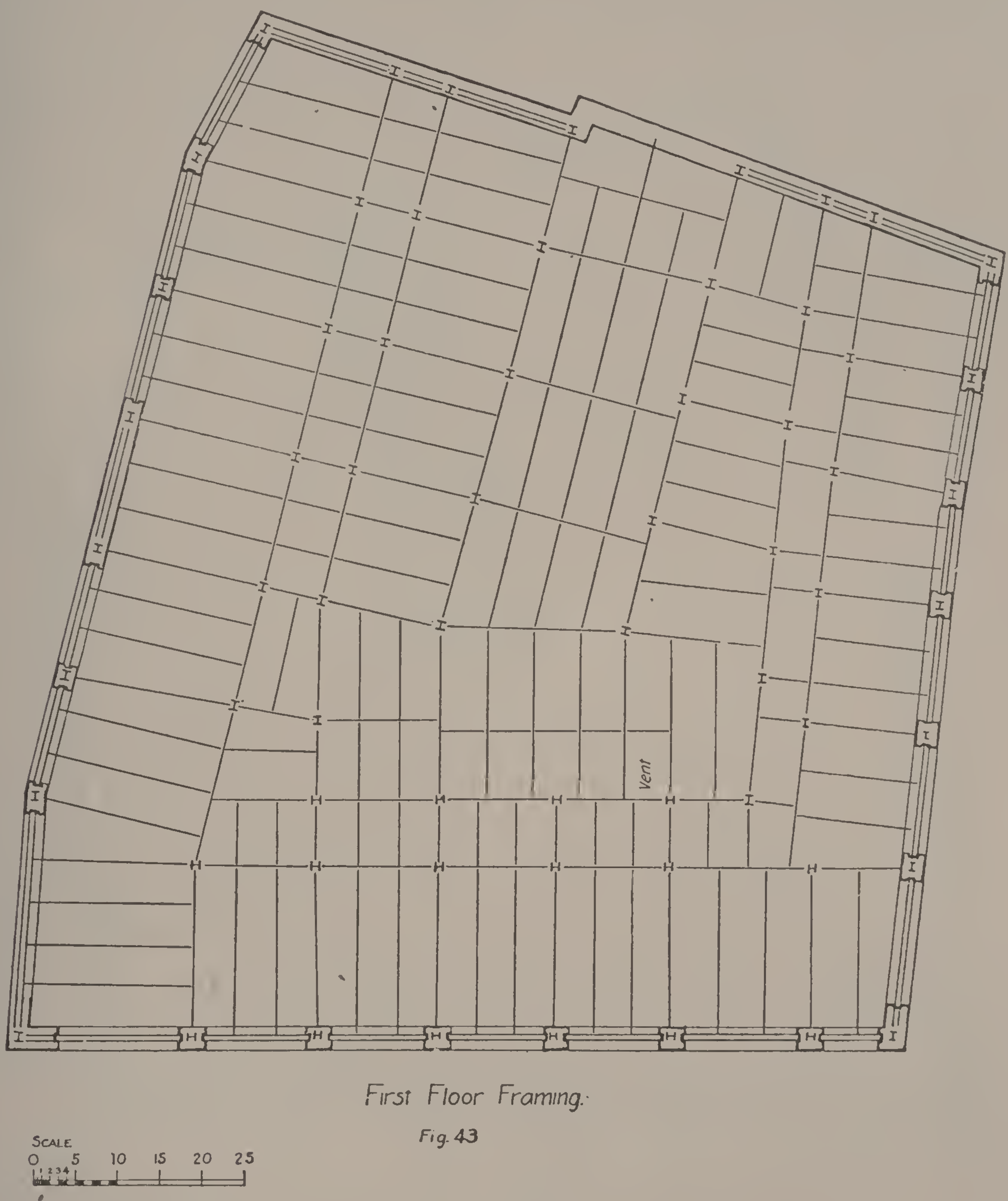
Position of Columns. The first step is the location of columns. These should always come in partitions, unless there is a large hall or like arrangement in which the columns form a feature. The position of the columns fixes, of course, the spans of beams and girders. A stiffer frame will result if the beams run



Typical Floor Plan
Fig. 42.

transverse to the longest dimension of the building. The girder spans should also be shorter than the beam spans, as otherwise excessive depth of girders will be required. In general, therefore, the shortest spacing of columns should be in the direction of the longest dimension of the building. The length of this space will be limited also by the allowable depth of floor system. For an

office building like the one in question, it is not desirable to use beams or girders over 12 inches deep, if possible to avoid it. With the above points in mind, we shall see what application can be made in this case.

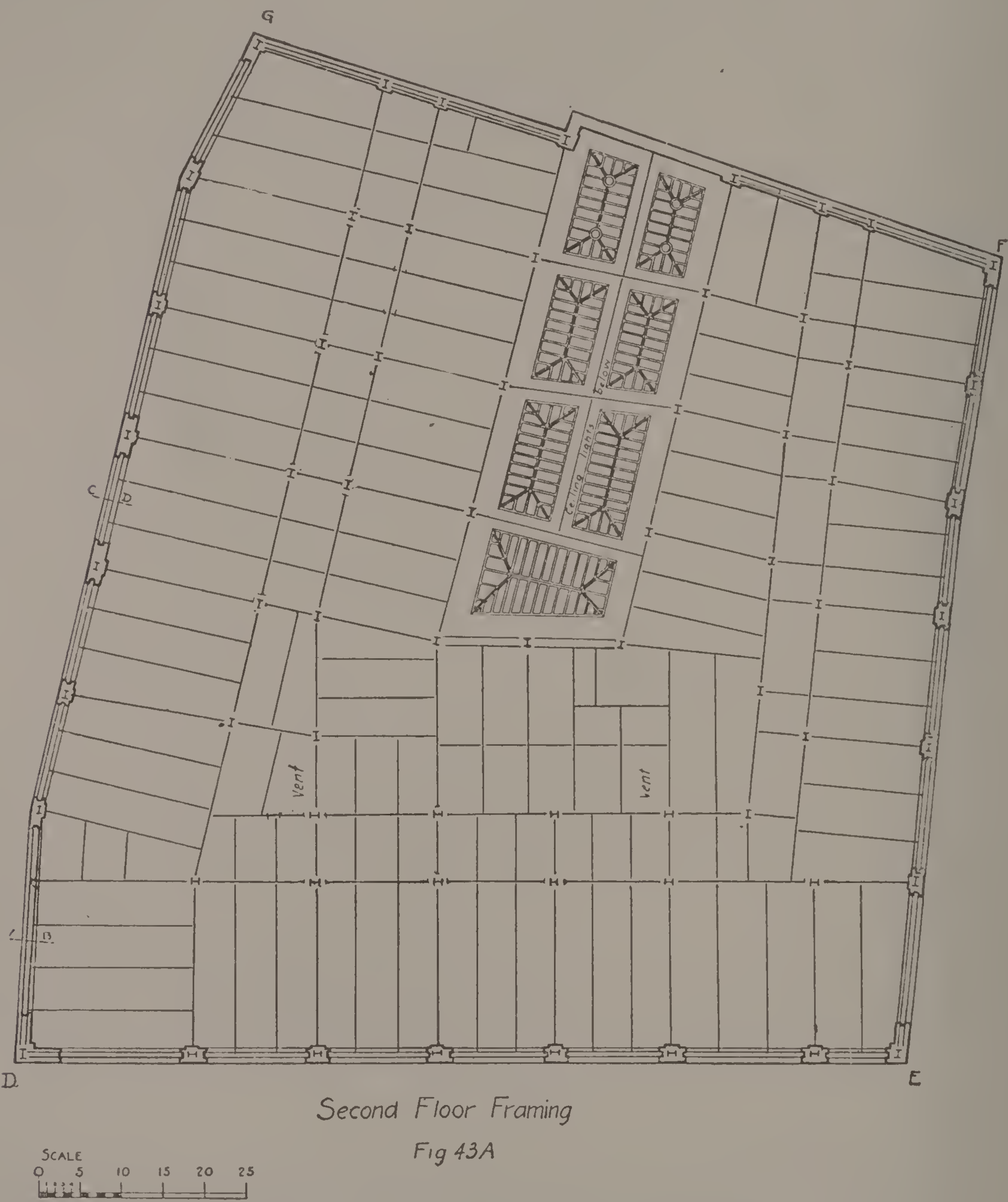


First Floor Framing.

Fig. 43

Columns cannot be located by a study of one floor plan alone, for the arrangement of rooms may vary from floor to floor so as to result in columns interfering with doorways or not coming in partitions in certain floors, though being well adapted to the conditions of some one floor. The natural method, therefore, is to take the typical floor plan, and then adapt the locations indicated

therein to the conditions on the other floors. Figs. 40, 41 and 42 show respectively the basement, first floor, and typical floor plans of an office building; and Figs. 43, 43A, and 44 show respectively the framing plans of the first and second floors and typical floor.



Second Floor Framing

Fig 43A

As will be seen, the lot is approximately of the same dimensions on each side. There is only one right angle, however, and one side has two very obtuse angles. The interior arrangement of the typical floor shows a line of offices on three sides, with corri

dors parallel on these sides, and an interior court. The effect of this court is to divide the building into sections whose longest dimensions are parallel to the exposed walls.

As before noted, it is an advantage for the sake of stiffness to have the girders run parallel with the long sides. It is further an advantage, and generally necessary, to have the girders of shorter span than the beams, and to have them come in partitions, as otherwise they would drop below the ceiling or necessitate a deep floor system. The first step, therefore, is to see whether the columns can be so placed as to meet all of these requirements. In the present instance it will be seen that in general this can be done by placing the columns at the intersections of office and corridor partitions or walls. This is, moreover, a desirable location for the columns, because with the thin partitions used in offices, a column cannot be fireproofed without exceeding the thickness of partitions, and it is not desirable to have a large column casing in the middle of a partition.

The next point to fix is the exact position of the column center with relation to the partitions and the direction of the column web. The corridor side should finish flush with the corridor partition, leaving the necessary casing to come in the offices. Therefore the center must come a little inside of the center of the corridor partition, and coincident with the center of the cross partition. As the greatest dimension of the column is generally in the direction of the web, it will be necessary to set this in less if the web runs parallel with the corridor partitions and with the girders. This is generally the best arrangement also for the framing for, in the upper sections of columns, the distance between the flanges of the columns might not be sufficient to allow the girder to frame into the web, while the beams, having a smaller flange, would take less room. An exception to the above consideration would be the case of double-beam girders, as will be explained later.

The location and position of the main interior columns having thus been fixed, the next thing is to locate any columns whose position is dependent on special features.

In this case, the corridor arrangement along the side E F at the end near D E makes it necessary to place this column out of the line of the others. On this account and to avoid excessive

loads on the girder framing into this column, an extra column is put in the partition between toilet and vent at this end.

In the exterior walls, columns of course have to be placed at each corner and also at the angles in the side A B C D. The other

Typical Floor Framing
Fig 44.

exterior columns naturally are placed at the intersections of office partitions with exterior walls, because here the piers in the walls will be the widest. The distance from the ashlar line to the center of wall columns will vary in accordance with the architectural details. There should never be less than four inches of masonry

outside of the extreme corner of column, and, if possible, there should be more.

Better protection is given the steel if the web is parallel with the face of the wall. Where the spandrel beams and lintels are very eccentric, however, this position results in an uneconomical section, since the weakest axis of the column is thus exposed to the greatest bending. Some designers, however, prefer to sacrifice economy in this regard to more efficient protection of the metal.

The columns thus having been placed according to the arrangement of the typical floor plan, the next step is to see if any changes are necessary to suit the conditions of the floors that differ from this plan, namely, the basement and the first floor. From a glance at the plan of the first floor, it will be seen that two of the columns come down in the main entrance in such position as to obstruct the passageway. It would be possible to change the position of these columns and make them conform to the first floor partitions. The results in the floors above, however, would not be so good, and therefore additional columns will be provided, supporting girders at the second-floor level to carry the columns above. A similar provision must be made for the wall column over the entrance.

The position of the columns thus having been determined, the girders follow by joining the centers of columns. The spacing of the beams will be determined largely by the system of floor arch to be used, except that, unless entirely impossible, a beam should come at each column in order to give lateral stiffness to the frame. If a terra cotta arch is to be used, the spacing should not be much over six feet at the maximum, and an arrangement such as shown would result. If a system of concrete arches is to be adopted, in which spans of eight or nine feet can be safely used, the beams between the two lines of girders on each side of the corridors may be omitted.

Certain other points should be noted in regard to this framing plan, as follows:

Columns should not be put at the front of elevators, as they cannot be fireproofed without interfering with the clear space of shaft.

Beams, if possible, should always be framed at right angles to girders, as oblique connections are expensive.

Beams should not frame off center of column if a little change in either column or beam can obviate it.

Columns on adjacent and parallel lines should, as far as possible, be opposite each other; that is, a beam framed to the center of one column should also meet the center of the next line of columns.

Spacing and spans of beams should be such as to develop their full strength.

Fig. 45 shows the wall sections and the resulting spandrel sections and wall girders. Not all the points that arise in such a framing can here be brought out; but from the foregoing the general method of treatment of such problems should be clear.

In buildings of a different character, many different and often more complex conditions will arise. The student, however, must always bear in mind that it is the duty of the designer to grasp fully the architect's details, and so to arrange his framing as to conform in all respects thereto, unless such details can themselves be changed more readily and to better advantage. It is essential for the designer to see not only what has already been determined, but what details will result when certain features are fully worked out; and in all his work the economy of design and framing, and the efficiency of the framework, should be kept constantly in mind.

The framing shown for this building is more especially designed for concrete floor arches. In cases where terra cotta arches are used, a somewhat different arrangement of columns would probably be made.

In the framing of floors and roofs, it is not always advisable to use the exact sizes and weights of beams that are theoretically required; there are often a number of practical considerations affecting the determination. As previously stated, standard sizes and weights should be used wherever practicable, as ordinarily these sizes are much more readily obtainable than others. If the general framing consists of standard sizes, and a few beams are so loaded as to require special sizes and weights, some change should if possible be made to avoid this, as to insist on the furnishing of a few beams of odd weights might cause serious delay in the delivery. In certain cases where it is of special advantage to make nearly all the beams of special weights, arrangements might be made for the delivery, provided the tonnage is large.

Beams, as far as possible, should be of the same size through-

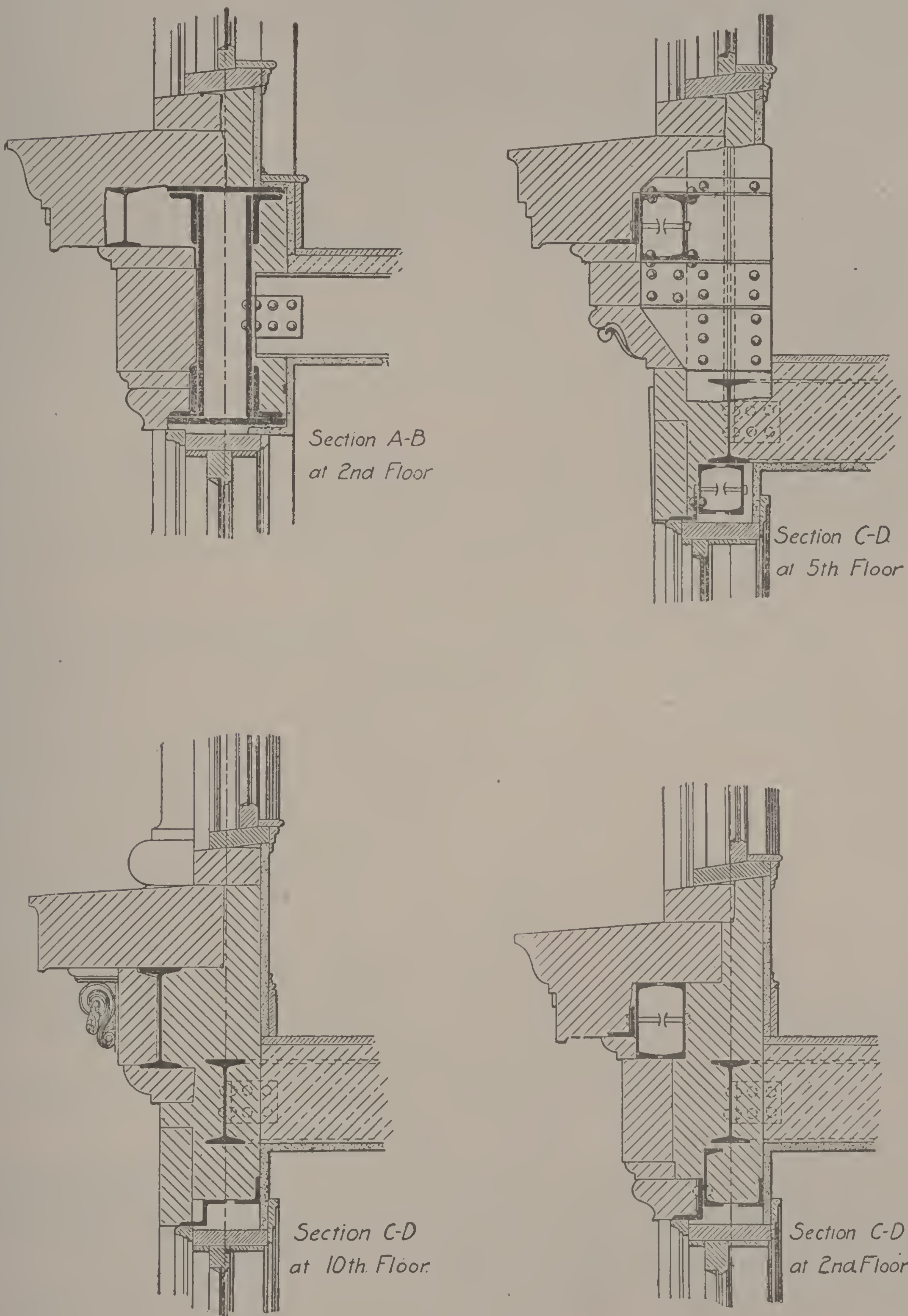

Section A-B
at 2nd Floor
Section C-D
at 5th Floor
Section C-D
at 10th Floor
Section C-D
at 2nd Floor

Fig. 45.

out a given floor, since for a level ceiling different depths of beam would require furring, or extra filling, or special arches. Where girders of short span carry the ends of heavy beams or girders, it is sometimes necessary to use an uneconomical section in order to get a sufficient connection. For instance, a 10-inch beam might be strong enough to carry a 15-inch beam; but the connection could not be made to a 10-inch beam, and therefore a larger sized beam or channel should be used. In general the girder should be of the same depth as the beam, or nearly so, unless the beam rests on top of the girder or is hung below it.

In some cases also — generally where small beams are used — the standard end connections are not sufficient, and it may be necessary to use larger sizes.

Other special conditions of framing are likely to arise, affecting the determination of sizes, so that the designer, in laying out the framing, should keep in mind the feasibility of making proper connections for framing the different parts.

When very heavy loads are carried by beams of short span, it is necessary to use a section that will have sufficient web area to prevent buckling. In such cases, the sizes of beams may be determined by this condition rather than by the bending moment caused by the loads. The tendency to cripple is greatest at the ends, and in order to determine the allowable fiber strain, a modification of the column formula as given below is applicable. The total shear should be considered to be carried by the web, and the combination of horizontal and vertical shear is equivalent to tension and compression forces acting at an angle of 45° with the axis of beam. The unsupported length in the formula, therefore, is the length between fillets on a line making 45° with the axis of beam.

Tie Rods. Tie rods should be spaced at distances not greater than twenty times the width of flange of floor beams.

The size of tie rods is generally $\frac{3}{4}$ inch diameter. An approximate determination of the required size can be made by use of the following formula giving the thrust from floor arches:

$$T = \frac{3\,W\,L^2}{2\,R},$$

where $T =$ thrust in pounds per linear foot of arch,

W = load per square foot on arch,

L = span of arch in feet,

R = rise (in inches) of segmental arch, or effective depth of flat arch.*

The spacing of the tie rods being known, the total strain on the rods is the thrust, as above, multiplied by the spacing. Dividing this by the safe fiber strain of 15,000 lbs. per square inch, gives the net area of rods, or the area at the root of threads, and thus determines the diameter of the required rod.

The spacing of tie rods is generally determined by providing one or more lines dividing into equal spacing the length of beams between connections or walls. The number of lines is determined by the necessity of keeping the thrust within the capacity of a certain size rod, or by the limit of twenty times the flange width.

FIREPROOF AND FIRE-RESISTING MATERIALS.

The functions of fire-resisting materials are threefold:

1. To carry loads.
2. To protect all structural steel.
3. To serve as noncombustible partitions or barriers.

The specific uses are, in general, the following:

1. Floor and Roof Arches.
2. Ceilings.
3. Partitions.
4. Protection for flanges and webs of beams and girders.
5. Protection of columns, doors, and shutters.

Fireproof materials, as generally used at the present time, comprise burnt clay in various forms, concrete, and plaster.

Fire-resisting materials, in general, comprise specially treated wood, certain kinds of paint, asbestos paper or other special kinds of paper, and metal-covered wood.

* NOTE. By "effective depth of flat arch" is meant the depth from top of arch to bottom of beam.

Plate II

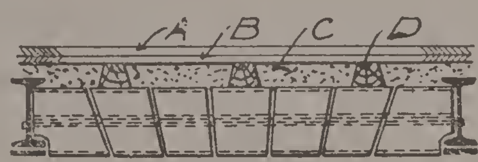


Fig.—46.
End Construction, Terra Cotta Arch.

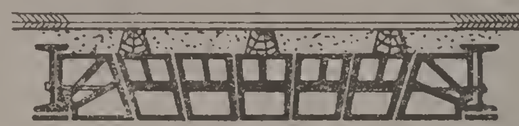


Fig.—47.
Side Construction, Terra Cotta Arch.

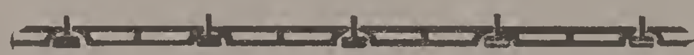


Fig.—48
Ceiling and Roof. Tile Block Construction.

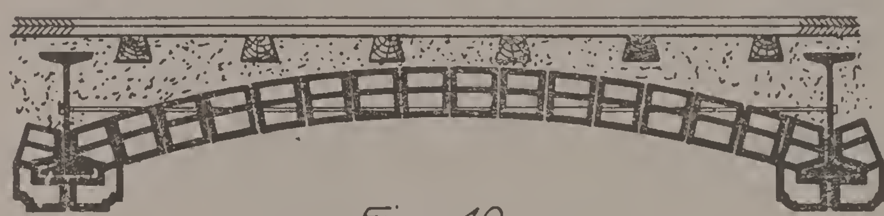
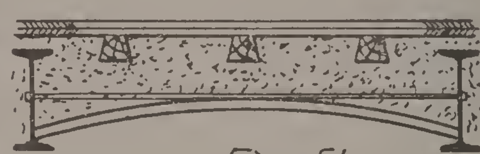


Fig.—49.
Segmental Terra Cotta Arch Construction.

Fig—50
Brick Arch Construction.



Fig—51.
Corrugated Iron Arch Construction.

FLOOR AND ROOF ARCHES.

Terra Cotta Floor and Roof Arches. Burnt-clay products include brick, porous tile, and hard or dense tile. The latter two are commonly called terra cotta.

The use of brick for arches between beams has, in building construction at least, become almost entirely obsolete. This is due largely to the saving in weight accomplished by the use of other materials.

When brick arches are used, the construction is generally of the type shown by Plate II, Fig. 50. There is a patented system employing brick, which is known as the Rapp system. The bricks here do not form a self-supporting arch, but are laid flat between metal ribs or bars that spring between the steel beams.

The use of burnt clay products in fireproof floor and roof arches and coverings of steel, is confined almost exclusively to terra cotta, and this is generally of the porous type.

Porous terra cotta is lighter and less brittle than the hard tile, with probably almost equal strength. It is made by mixing straw with the clay, the mass, when burned, being thus left porous. It also has the advantage that it can be nailed into, this being especially important in roof and partition blocks.

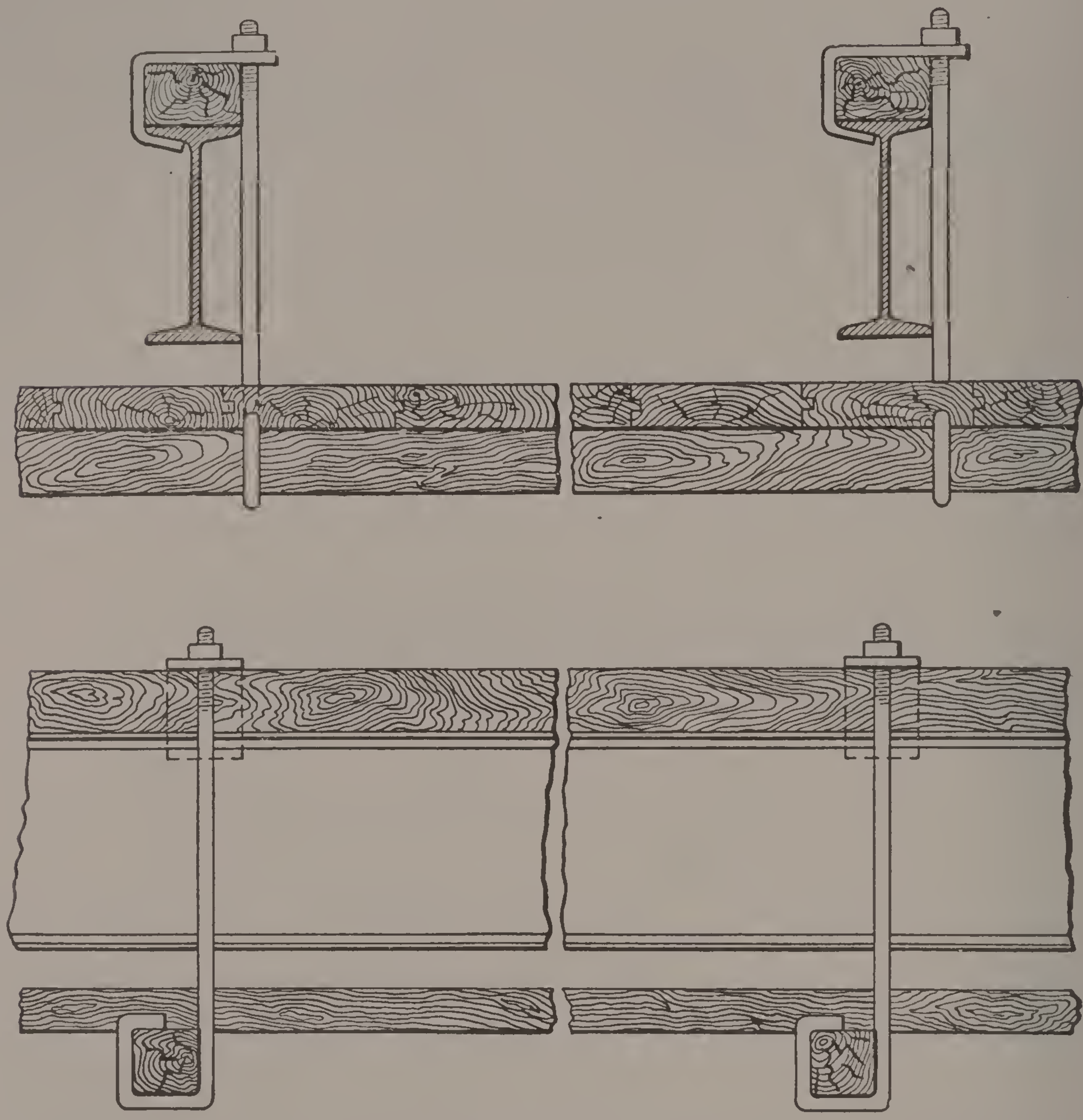
Terra cotta arches were formerly laid up exclusively with the ribs running parallel to the beams, this construction being known as **side construction.** Tests have shown, however, that the arch is stronger when laid with the ribs at right angles to the beams; and this practice, which is known as **end construction,** is now generally followed. Figs. 46 and 47 (Plate II) show these two constructions.

An inspection of these cuts will show that the arch consists of a key, voussoirs, and skew-back, shaped similarly to the practice in masonry arches. It should be noticed that in side construction a special-shaped skew-back is required, which is not the case in end construction. Also notice that the piece protecting the flange of the beam is separate from the arch, this being a simpler plan than to shape the skew-back so as to cover the flange.

The arch is generally two inches lower than the bottom of the beam, thus coming flush with the flange piece and giving a flush surface for plastering.

The construction of wood screeds and top flooring shown is almost always used, although other forms could be adopted. The filling between the screeds should always be a cement concrete, although cinders instead of stone may be used.

The method of supporting the centers for these arches is shown by Fig. 52. This construction allows the centers to be readily removed after the arch has set.



CENTERS FOR TERRA COTTA FLOOR ARCH.

Fig. 52.

The practice, in general, is to set the floor arches, from the lower floors up, after the steel frame has been carried several stories in advance. Centers used in the lower floors can be used in the upper floors unless the work progresses very rapidly.

Roof arches, on account of the pitch of the beams, have to be furred down to give a level ceiling. Terra cotta blocks may be used for this purpose, as shown in Fig. 48. It is, however, quite

as common, even where terra cotta floor and roof arches are used, to form the furred-down ceilings of small channels or angles covered with some form of wire lath. A construction of this sort is shown in Fig. 53.

For ordinary loads it is not usually necessary to calculate the

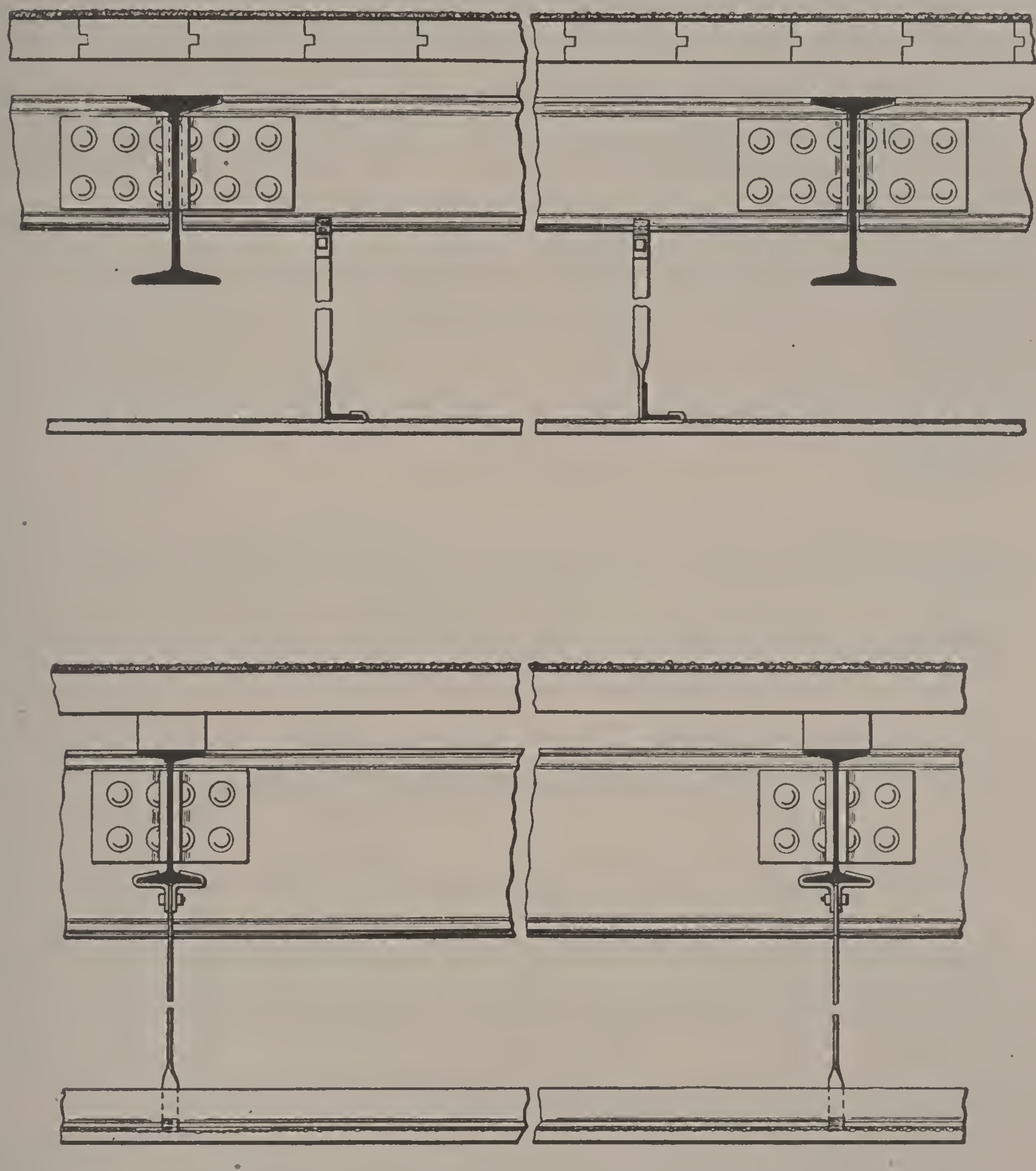


Fig 53

depth of terra cotta arch required. The spans are kept within certain limits, and for such limits the proper depth of arch has been well determined.

The following spans and depths of arches represent the accepted practice:

When it is desired to use terra cotta construction for heavy loads, such as in stores and warehouses, a segmental arch is used, generally 4 inches or 6 inches in thickness. The filling above the arch consists of concrete, either of stone or cinders. This construction is illustrated in Fig. 49. While greater spans are sometimes used, the best practice does not exceed about 8 feet, and is preferably limited to 6 feet.

Guastavino Arch. This is a dome or vault system especially adapted for long spans where a flat ceiling effect is not essential, as in churches, libraries, halls, etc.

The construction consists of several layers of hard tile one inch thick, laid breaking joints. The number of layers varies with the conditions, but generally does not exceed four. The rise of the dome is ordinarily not great; and it rests either between walls or, in some cases, on heavy girders. The tiles are usually set in Portland cement, except that the first course is set in plaster in order to obtain a quick set and to dispense with a certain amount of centering.

This system is almost always installed under a guarantee from the company controlling the patents, as to its efficiency and adaptability to the conditions of the special case in hand.

Concrete-Steel Floor and Roof Arches. The types of concrete and concrete-steel arches are becoming more numerous each day, and only a few will here be discussed. They may be separated primarily into flat arches and segmental arches. In most of the systems of the flat-arch construction, the action is essentially that of a beam of concrete in which metal is embedded on the low side to increase the tensile strength, since concrete is not as strong in tension as in compression. In a few of the systems, however, when special-shaped bars are used at short intervals, the effect is more that of a simple slab of concrete supported by these bars, which act as small beams between the main floor beams.

In the segmental form of concrete construction, the metal, where used, is generally intended more as a permanent center for forming the arch and for supporting it until the concrete has fully set, when the concrete is considered as taking the load independently of the steel center.

Plate III shows types of Expanded Metal Floor Construction. Fig. 54 shows System No. 9, which can be adapted to long spans. It is not the general type of this form of construction, however, as

the types shown below are generally considered more economical. In calculating the weight of the construction, the arch should be figured

For Systems Nos. 3 and 5.

A.B.C.E H. same as for No. 9.
D = slab of cinder concrete.
K = angles for support of ceiling.
L = expanded metal ceiling.
M = hangers securing ceiling angles to beams.
N = slab of cinder concrete on expanded metal, protecting webs of beams.
O = solid concrete haunch protecting web of beams.

For System No. 7.

A.B.C.D.E. same as for No. 9.
O = solid concrete slab.

For System No. 9.

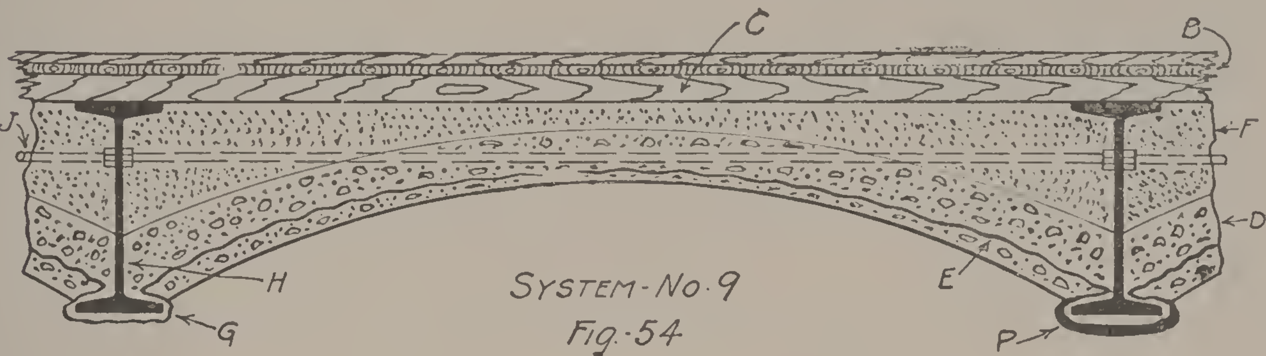
A = top floor.
B = under floor.
C = wood screeds or sleepers.
D = arch, cinder concrete.
E = expanded metal sheet.
F = cinder concrete filling around and under screeds.
G = expanded metal wrapping of flanges to receive screened plaster as shown at P.
H = main floor beam.
J = tie rods.

For System No. 8.

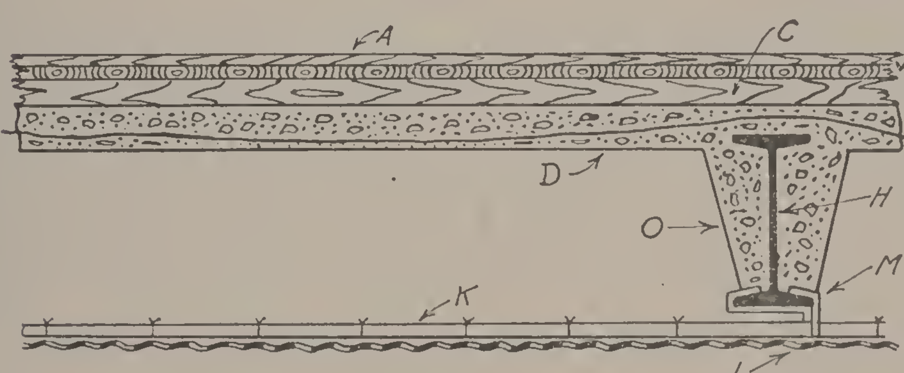
A.B.C D.E F G.H. same as for No. 9.

For System No. 4.

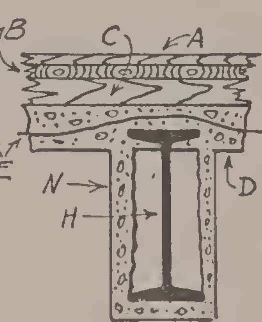
A.B.C D E.H. same as for No. 9.



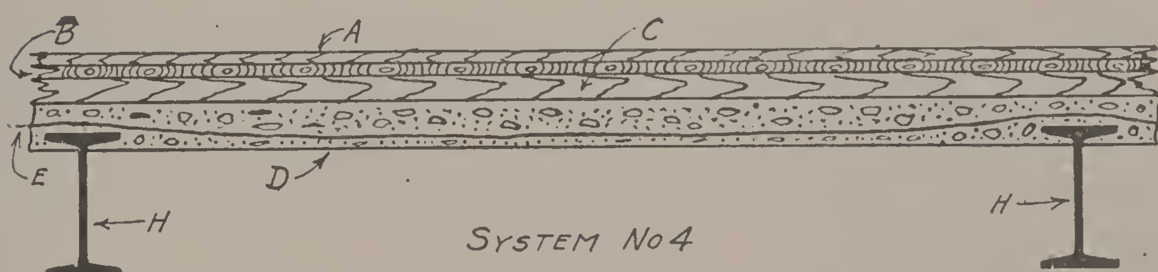
System-No 9
Fig-54



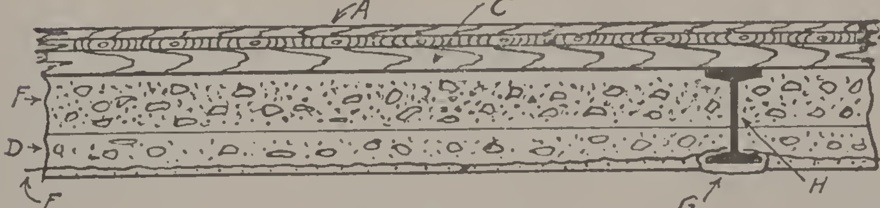
System-No 3
Fig-55



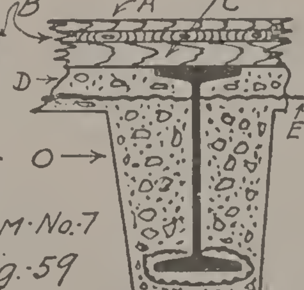
System-No 5
Fig-56



System No 4
Fig-57



System-No 8
Fig-58



System-No.7
Fig-59

separately from the filling above, as the weights of these are different. The same remark applies to all systems of concrete construction.

Fig. 55 shows System No. 3, with a furred-down ceiling to give a level effect. This ceiling is not a necessary part of the construction, and is often omitted. The space between ceiling and floor slab is available for running of pipes, wires, etc.; and, to avoid punching of beams when such use is made of this space, the ceiling is dropped below the flanges of beams far enough to allow the passage of pipes, wires, etc.

This system is the one generally employed for long spans and heavy loads, as it gives the most substantial protection to the steel, and has certain elements of strength not possessed by the other systems, as follows: The haunches, besides protecting the webs and flanges of beams, shorten in effect the span of floor slab, and stiffen the floor beams against side deflection. The sheets of expanded metal can be made in effect continuous over all floor beams, and, because of this, the whole construction from wall to wall acts together, and has the advantage of a continuous beam over a number of supports. While it is impossible to state exactly what this advantage amounts to, on account of the uncertainty of actual conditions conforming to the theoretical assumption, it is probably safe to assume that the strains in the floor slab of a construction having this continuous feature would not be more than three-quarters as much as if the slabs were discontinuous. It should be noted in the above system, that if the furred ceiling is omitted the lower flanges of the beams are protected in a manner similar to that shown for System No. 7.

System No. 5, illustrated by Fig. 56, differs from System No. 3 only in the method of protecting the beam. As will be seen, all the strength afforded by the haunch is lost by this construction, and, as will also be seen later from results of tests, the protection is much less fireproof.

Fig. 57 shows System No. 4, which differs from System No. 3 only in the entire omission of protection to floor beams. This system is therefore only semi-fireproof, and in event of fire in the story below would not be to any degree fireproof. It is sometimes used with a fireproof suspended ceiling, but, as will be noted further on, tests of such ceilings have shown them to be of questionable value as efficient fire barriers.

Fig. 58 shows System No. 8. This system is chiefly adapted

to light loads on moderately long spans where the beams are in general not over 8 inches or 9 inches deep. In such cases, where a flush ceiling is desired, it is sometimes more economical than some of the other systems with suspended ceiling.

It has the disadvantage from the standpoint of strength, that the load all comes on the lower flanges of beams, and further, that all continuous effect of slabs is lost.

Fig. 59 shows System No 7, really a modification of System No. 3, in which, in order to save depth, the floor slab is flush with the top of the floor beams.

This system also has the disadvantage of loss of continuous effect. In all the above systems, the more common spans are from 5 feet to 8 feet. The company controlling the patents, however, claim to be able with safety to adapt the construction to longer spans, even under heavy loads.

In these systems, as well as in all others where a cinder filling is used on top of the floor slab, the filling should contain some cement, as otherwise the unneutralized cinders are likely to cause corrosion of the steel.

The depth of floor slab varies with the load and the span, but is ordinarily 3 inches or 4 inches for loads under 200 lbs. and spans of about 5 feet.

Plate IV illustrates types of the Roebling system of fireproof floors. Fig. 60 shows System A, Type 1, which consists in general of a wire center sprung between the bottom flanges of floor beams, and upon which is deposited cinder concrete in the form of a segmental arch whose top is flush with the top of floor beams.

The strength of this system is considered to be simply that of the concrete arch, the wire center being intended merely for the support of the concrete until it has set, and for a permanent center upon which plastering may be applied directly if a level ceiling is not desired. This construction, Type 2, is shown in Fig. 61. It is further claimed for this wire centering, that it facilitates the more rapid drying out of the concrete on account of exposing both surfaces to the air and allowing the surplus water to drip through.

Fig. 62 shows System B, Type 1. This is a flat arch construction in which the steel members are bars spaced generally about sixteen inches center to center, the concrete slab being

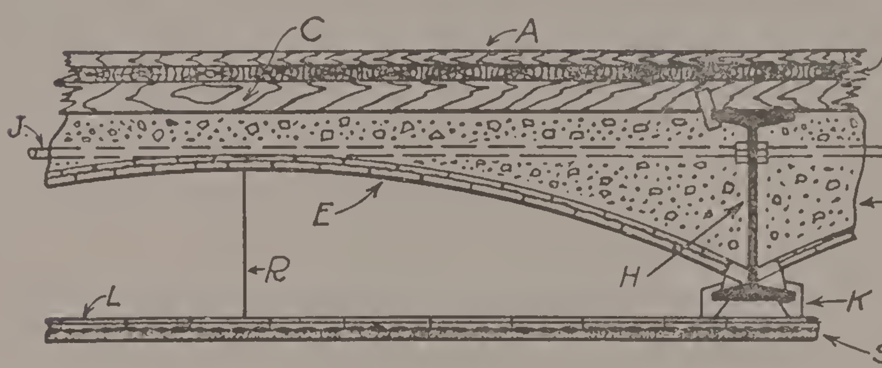
usually $3\frac{1}{2}$ inches thick. The bars are tied transversely by wire rods spaced about 24 inches on centers and serving to keep the bars in place.

Plate IV *Types of Roebling System of Floor Construction*

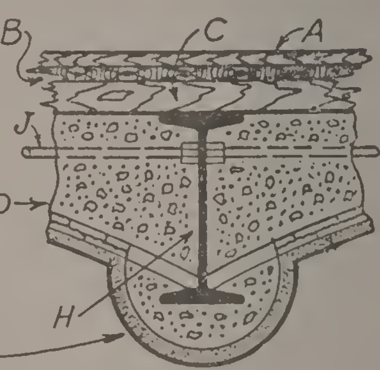
FOR SYSTEM B.
A.B.C.H.R.S.K.L same as for Sys. A.
D = cinder concrete floor slab.
M = flat bar 2″ x $\frac{3}{16}$″ or 2″ x $\frac{1}{4}$″.
N = solid casing of cinder concrete.
O = $\frac{3}{4}$″ x $\frac{3}{16}$″ flat bar.

Note:—Items R.K.O apply to Type 1 only.
Item L applies to Types 1 and 4 only.

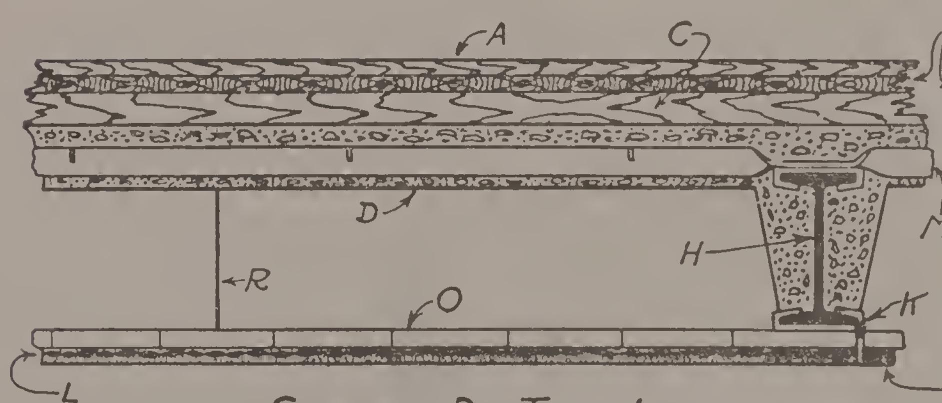
FOR SYSTEM A.
A = top floor.
B = under floor.
C = wood screeds or sleepers.
D = cinder concrete arch.
E = steel rod ($\frac{7}{16}$″ or $\frac{9}{16}$″) woven into wire lathing.
J = tie rods.
H = main floor beams.
S = plaster ceiling.
R = supporting wire.
K = clamp supporting ceiling.
L = steel rods woven into wire lathing.
Note:—Items R.K.L apply to Type 1 only.



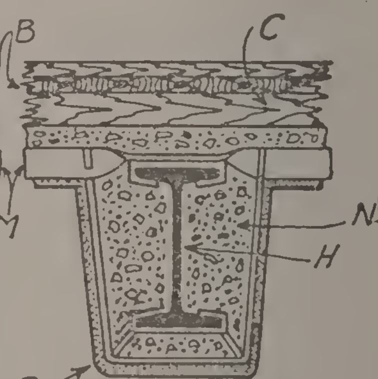
SYSTEM-A - TYPE-1
Fig-60



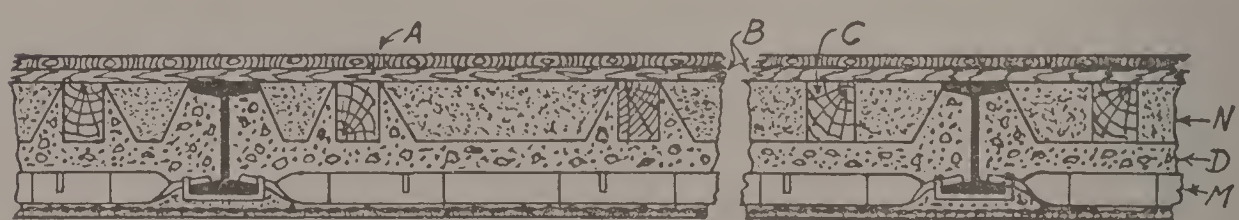
TYPE-2
Fig-61



SYSTEM-B - TYPE-1
Fig-62



TYPE-2
Fig-63.



SYSTEM-B - TYPE-4
Fig-64

Fig. 63, Type 2, shows the construction when the suspended ceiling is omitted. This suspended ceiling does not always have the bars shown by Fig. 62, but for short spans has simply the wire cloth stiffened by rods woven into it.

Fig. 64 shows System B, Type 4, in which the floor slab rests on the lower flanges, and the cinder filling is flush with the top of floor beams. This system makes some saving in depth, but is open to certain objections, one being the disadvantage from the standpoint of strength of resting the slabs on the bottom flanges, and another the absence of all protection or covering for the top flanges of beams.

The practice of the company controlling the patents is to deposit the concrete without any tamping such as is ordinarily done in the other systems. The claim is made that this method insures lightness and preserves its porosity, being thus rendered less subject to the effects of changes of temperature, either of the outer air or under exposure to fire and water.

As will be noted later, Professor Norton advocates tamping of concrete to eliminate the possibility of voids, which he shows to be always productive of corrosion of the steel.

Plate V shows types of the Columbian system of fireproof floors. This is a flat arch system, in which the action of the floor slab is that of a concrete beam with imbedded steel bars.

No continuous effect such as is had in some of the other systems exists in this construction, except as the whole construction of girders and their casing may be considered as acting together. The connection of the bars to the floor beams, and the concrete being finished flush with tops of beams, make the slab, considered by itself, discontinuous.

In the systems previously described, cinder concrete is almost invariably employed. In this system, however, the use of stone concrete is the prevailing practice.

The different types vary only in the size and spacing of the imbedded bars (and consequently in the thickness of the concrete slab) and in the connection of these bars to the beams. This connection is made either by means of small angles bolted to the webs of floor beams similarly to regular beam framing, or by means of hangers resting on the top flanges of beams. The former construction is used only when special stiffness of the frame is required, as in high building construction.

The thickness of slab is generally $1\frac{1}{4}$ inches more than depth of bar. The spacing of bars and of beams varies with the required

loads. The different cuts shown (Figs. 65, 66, and 67) give reasonable limits. In any case of special loading, however, or of spans exceeding 8 feet, tests should be made in accordance with the required conditions.

The explanations given on the plate, in connection with the above, should make the construction clear. It is the practice, in using this system, to have slots in the brick walls at the level of the floor slabs, and the bars and concrete slabs are then imbedded in these slots. This gives a good tie for the walls, and obviates the necessity of channels against the walls to take the floor construction.

In all calculations of the weight of dead loads where this system is used, the difference in weight between cinder concrete and stone concrete must be noted.

Figs. 69 and 70 show the Ransome system of floor construction. This is one of the oldest forms of concrete-steel construction, and is used in various modified forms to suit different conditions. It consists of steel rods imbedded in the tension side of the concrete; these rods run transversely to the beams, and are tied longitudinally by other rods. In some forms of this construction, steel girders and beams are replaced by deep concrete beams with heavy rods imbedded therein, and tied at intervals by U-shaped rods. The use of rods in the concrete makes possible many varied forms of construction, but special knowledge of the subject is required to design such forms properly.

The use of concrete and concrete-steel arches cannot as yet be considered to be very general. They are of comparatively recent introduction; and although, in the aggregate, they may now be said to be extensively used, there is as yet no one form recognized as standard.

The Building Departments of all cities have required special and severe tests of full-sized arches to be made before allowing any of the types to be used in construction. Their use is undoubtedly growing, and perhaps more especially in warehouses and buildings of heavy construction. There are certain features not possessed by any of the concrete systems; and this fact, probably, to a great degree explains the more general use of terra cotta in office buildings.

Plate V.

Types of Columbian System of Fireproof Floors.

For Systems Nos 2 and 3.

F = ribbed bars imbedded in concrete (2" and 2½")

J = stirrups.

For System No 3.

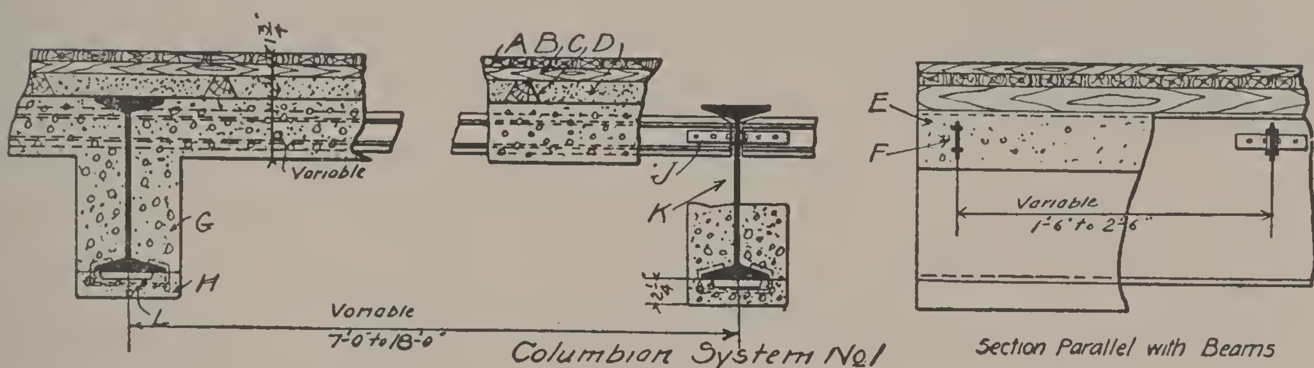
G = concrete ceiling slab

H = 1" ribbed bar imbedded in concrete

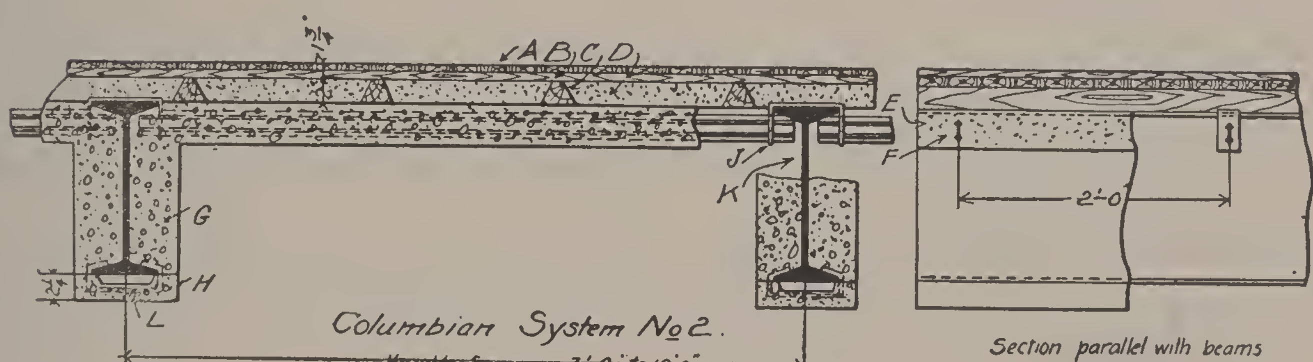
Note; Items not mentioned above are same as for System No 1.

For System No 1

A = top floor.
B = under floor.
C = wood screeds or sleepers
D = cinder concrete filling
E = concrete floor slab (thickness = 1½" + depth of bar)
F = ribbed bar (3½, 4¾ or 5") imbedded in concrete
G = casing of floor beams.
H = slab protecting flanges of floor beams.
J = angles connecting bars to floor beams.
K = main floor beams
L = concealed anchors holding slab

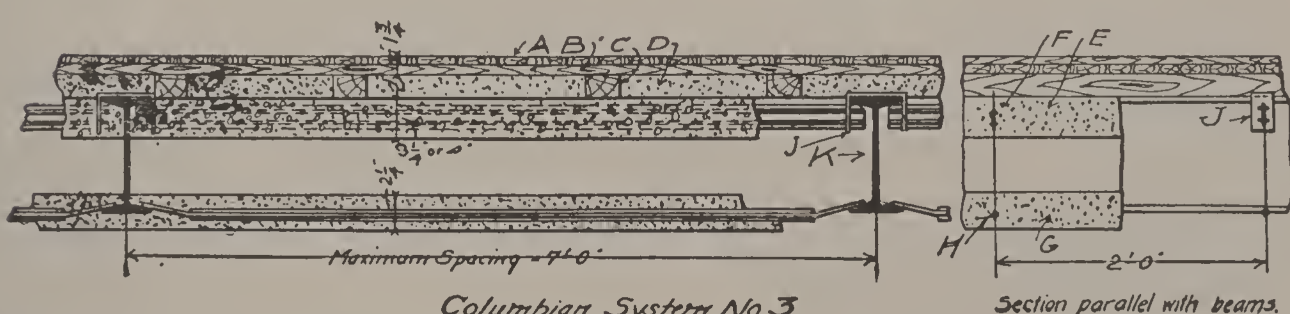


Columbian System No 1
Fig. 65



Columbian System No 2.
Fig. 66.

Connection of Columbian floor slab to brick walls
Fig. 68.



Columbian System No. 3
Fig. 67.

As noted previously, an important feature in buildings not having heavy masonry walls is lateral stiffness. This lateral stiffness is secured to a considerable degree by the floor construction, which serves to tie together all parts of the framing at each floor level, and also to distribute the lateral strain throughout the whole.

A floor construction which fills the whole depth of the beams is therefore better calculated to perform this function than one that is comparatively thin, as are nearly all the concrete systems. Another important consideration concerns uniformity of material. Porous terra cotta, like brick, is easily inspected, and a nearly uniform product can thus be secured. The strength of concrete and of concrete steel, however, depends very largely upon the use of proper materials and their proper mixing and laying in place. Much greater variation is here likely to occur, and consequently a greater or less uncertainty as regards uniformity of results must exist. Another point to be considered is the necessity of having the concrete or concrete-steel system installed by the company controlling it, this resulting from the patents covering each form of construction. A still further advantage is the flush ceiling given by the terra cotta blocks.

There are, however, numerous points to be cited in favor of many of these systems. The general trend of investigation and discussion is toward a better understanding of the possibilities of concrete steel in general, and this will not unlikely result in the future in its more extensive use.

It is not the general practice of individual designers to calculate the required depth of slab in the above systems, except in the case of unusual loads and spans; but, as in the case of the terra cotta systems, tests have largely determined the limits of spans for various depths and loads. As concrete arches are used for heavy as well as light loads, however, there is need of more exact data than is at present available to determine their capacities under different conditions.

It cannot be said to be conservative practice in any of these systems, much to exceed eight feet in the span of the arches. The uncertainty of the quality of the concrete when cinders are used, and the uncertainty of set in the deeper slabs, together with

numerous other circumstances likely to affect the uniformity of the product, make it important to keep within this limit.

As will be seen from the illustrations, nearly all the concrete systems require furring down to give level ceiling.

Tests of Floor and Roof Arches. The most severe test of all forms of floor arch is their exposure to fire and water when under load. As above stated, one of the functions — and a very important one — of all fireproof materials is to protect the steel; for, if the covering falls off, leaving the steel members exposed to fire, the steel frame will soon fail. None of the materials used — terra cotta or concrete in its various forms — are of themselves

Types of Ransome Floor Construction

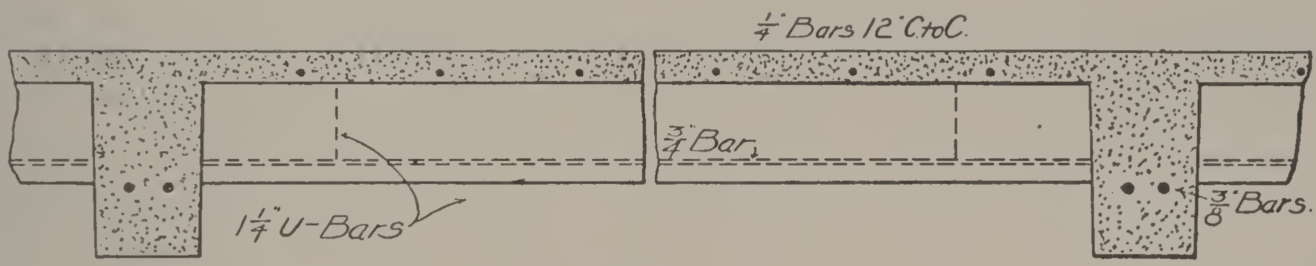


Fig 69

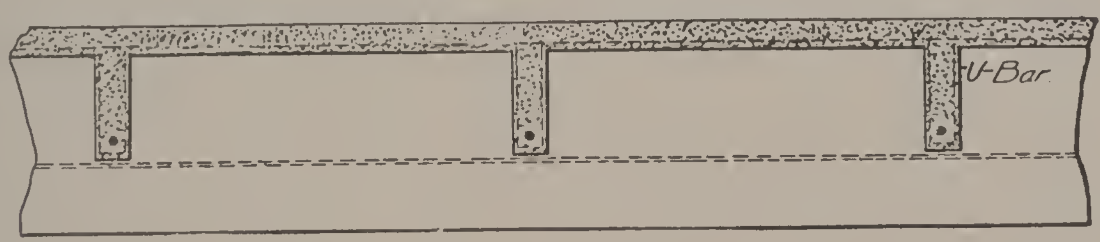


Fig 70.

combustible. Failure, when it occurs, is generally due to expansion and contraction caused respectively by the intense heat and by the chilling effect of the stream of water, and to the force of the stream knocking off pieces that become loosened. All of the systems in general use have been subjected to very severe tests of this character without collapse, before being accepted by the different Building Departments; and it is probable that when failure occurs in actual building fires it is due to constructive defects, there having been less careful construction than was used in the tests.

If only a small portion of the covering becomes detached, the whole adjacent construction is seriously endangered. It will be seen from the above that failure is more likely to start from detachment of the covering of beams, girders and columns, than in the body of the arch, and such covering should be as substantial as possible. For this reason, haunches or a solid filling protecting the beams and girders are preferable to wire lath wrapping the same.

Tests by the New York City Building Department on floors having suspended ceilings of wire lath and plaster, resulted in these ceilings being entirely destroyed. Tests of different floor systems having rolled shapes, such as T bars or special-shaped bars, imbedded in the concrete slabs, showed less deflection under loading than when a mesh of wire rods was used.

The method of testing floor arches is as follows: A brick furnace is built, having a large combustion chamber, the top being of the floor construction to be tested. This arch is loaded with a load generally four times that specified. Measurements of deflections due to the stress are taken before and after exposure to the fire. During this exposure, which generally lasts several hours, a temperature of from 2,000° to 2,500° is constantly maintained. After some time a stream of water from a fire nozzle is played on the arch, thus reproducing as nearly as practicable actual conditions.

After the test, the load is removed to see how great the permanent deflection is. It is important in all loading tests to have the load applied over a definite area, so that the exact load per square foot can be determined, and to avoid all possibility of any portion of the load bearing on the beams instead of on the arch.

The results of some tests made under different conditions are here given:

A fire and water test on a concrete expanded metal floor composed of $6\frac{1}{2}$ inches of concrete mixed in the proportion of 1 part Portland cement, 2 parts sand, and 5 parts cinders, showed the following results:

The slab was of a type similar to that shown in Fig. 55, the beams being 20-inch beams and spaced about 12 feet center to center, with the span of the beams about 17 feet 9 inches.

The slab of concrete was loaded with 400 lbs. per square foot, under which it deflected .30 inch. Under exposure to fire the deflection increased to $2\frac{1}{8}$ inches, and when the test was completed remained about $3\frac{5}{8}$ inches.

A portion of the under side of the concrete was knocked off by the stream of water.

A test under practically the same conditions as above was made of the Columbian system. The type was of the general form shown by Figs. 65 to 67. The spans were the same as above. The slab was $8\frac{1}{4}$ inches in depth, composed of 1 part Portland cement, $2\frac{1}{2}$ parts sand, and 5 parts broken stone. The bars were 5-inch bars, spaced 2 feet center to center, and fastened to the beams by angles.

A portion of the girder covering consisted of mackite blocks plastered, another portion consisting of 2-inch cinder concrete, the latter being the regular construction. There were also two 8-inch I-beams set up, one covered with cinder concrete to 9 inches × 13 inches; the other covered with hollow bricks to 12 inches × 16 inches, giving 4 inches covering.

The floor was first loaded with 1,000 lbs. per square foot, under which it deflected $\frac{3}{8}$ inch. The load was then reduced to 400 lbs. per square foot, and the fire test commenced. This lasted for two and one-quarter hours at a maximum temperature of 1,700°. A stream of water was then applied for $4\frac{1}{2}$ minutes, and afterwards another fire test given of 38 minutes and a second stream of water applied. The floor, at the end of the test, showed a deflection of $1\frac{1}{8}$ inches. The cinder concrete beam and column coverings were not materially damaged. The mackite covering was entirely stripped off, and the hollow brick column covering badly damaged. No apparent injury occurred in the floor slab. After this test the floor was loaded up to 1,650 lbs. per square foot, at which point the walls of the test house made it necessary to stop. The net deflection under this load was $1\frac{5}{8}$ inches. A few cracks appeared in the ceiling under this load, most of them being parallel to the bars.*

Numerous other tests of expanded metal floors on shorter

*NOTE. A detailed report of these tests is given in *Engineering News*, June 27, and November 21, 1901.

spans have shown satisfactory results. For spans up to 8 feet and loads under 200 lbs. per square foot, which are the ordinary conditions, the cinder concrete shows safe results. Beyond these limits special tests should be made in each case.

A valuable review of the effects of a practical fire test on terra cotta and concrete floor construction, is given in the discussion bearing on the fire that occurred in the Horne Building, Pittsburg, Penn., May 3, 1897, which was published in *Engineering News*, May 20 and 27, and July 1 and 15, in that year. An account of a second fire which occurred on April 7, 1900, is published in the same periodical under dates of April 12 and April 26, 1900.

The New York Building Department conducted a test on three arches of the Guastavino type, each 3 feet in length. The spans were 6 feet, 10 feet, and 12 feet. The 6-foot span was composed of 2 courses of tile, making a thickness of $2\frac{1}{2}$ inches; the 10-foot span, of four courses, giving 5 inches thickness; and the 12-foot span, of three courses, with a total thickness of $3\frac{3}{4}$ inches. All were leveled up with concrete. The 6-foot span carried 2,500 lbs. per square foot, and showed a maximum deflection of .13 inch. The 10-foot span carried 3,600 lbs. per square foot, with a deflection of .19 inch. The 12-foot span carried 3,125 lbs. per square foot, with a maximum deflection of .32 inch.

This was a simple loading test with no application of fire and water.

Tests of porous terra cotta hollow tile arches have not been so numerous, especially under fire exposure. Table XIII gives the results of a series of tests to determine breaking loads of different arches, and is taken from the "Transactions" of the American Society of Civil Engineers, Nos. XXXIV and XXXV, of 1895 and 1896.

In terra cotta arches as in concrete arches, great variations in strength will result from varying degrees of thoroughness in construction. These arches should always be set in cement and carefully keyed, and the use of broken blocks should be avoided. Settlement in arches of this type often results in cracks in tile or mosaic floors.

TABLE XIII.

Breaking Loads of Hollow Tile Arches.

Depth of Arch.	Rise.	Span.	Length.	Total Load.	Load per Sq. Foot.	Total Horizontal Thrust.	Horizontal Thrust per Ft. of Arch.	BLOCKS. Style.	BLOCKS. Material.	Character of Load.	Manner of Laying Joints.
Ins.	Ins.	Ins.	Ins.	Lbs.	Lbs.	Lbs.					
6.	3.5	60	48.	13750	688	29474	7369	E	Hard	Dis.	Port.
7.5	5.	46	11.5	9000	2452	10367	10818	"	"	"	N. M.
7.5	5.	60	35.2	11250		33750	11505	"	"	Cen.	Port.
7.5	5.	60	36.5	13000		39000	12822	"	Porous	"	"
8.	7.	60	38.25	14500		31071	9747	"	"	"	"
8.	7.	60	38.25	15750		33750	10588	"	Hard	"	"
12.	10.	60	41.	16400		24600	7200	"	"	"	"
12.	8.75	60	10.	3100		5314	6377	"	"	"	N. M.
12.	9.	60	10.	5000		8333	10000	"	"	"	"
12.	9.	60	10.	15100	3630	12583	15100	"	"	Dis.	"
12.	9.5	60	10.	2500		3947	4736	"	"	Cen.	......
8.	5.5	46	11.5	2500	681	2614	2727	S	"	Dis.	N. M.
8.	5.	45	11.5	1300	362	1463	1526	"	"	"	"
8.	6.	60	36.	10000		25000	8333	"	"	Cen.	Port.
8.	5.	60	36.	5700	380	8550	2850	"	"	Dis.	"
8.	5.	60	12.	3500	700	5250	5250	"	"	"	N. M.
8.	5.5	60	12.	10000	2000	13636	13636	"	"	"	"
8.	5.5	60	12.	2500		6818	6818	"	"	Cen.	"
8.	5.5	60	24.	9950	995	13568	6784	"	"	Dis.	"
8.	5.5	60	24.	2500		6818	3209	"	"	Cen.	"
10.	7.5	60	36.	13500	900	13500	4500	"	"	Dis.	Port.
10.	8.	60	37.	14500	940	13594	4408	"	"	"	......

In the above table the following abbreviations are used: "E"—end construction; "S"—side construction; "H"—hard clay: "Porous"—porous terra cotta; "Dis."—distributed load; "Cen."—concentrated load at center; "Port."—Portland cement; "N. M."—no mortar.

The loads per square foot in the above table were obtained by dividing the total load by the superficial area of the arch in square feet. The horizontal thrusts were obtained by the regular formulæ; for central loads these are double the thrusts for distributed loads of the same weight.

SELECTION OF SYSTEM.

Not any single system, probably, would be used in all cases even if the designer were to choose without any conditions affecting his selection. Some systems are naturally better adapted than others to certain conditions. Practically there are always a number of considerations affecting the choice. No attempt will be made here to specify to what conditions certain systems are better adapted than others, as this is largely a matter of judgment at the present time. The considerations in general, however, are as follows:

Light or heavy live loads; dead weight of construction, and con-

sequent spacing of beams and span of arches; necessity of lateral stiffness in floor system; possibility of using paneled ceiling, and consequent increase of clear height story between beams; necessity of flush ceiling, and comparative advantage of solid floor system and furred-down ceiling; protection afforded webs and flanges of beams and girders by different systems; possibility of omitting tie rods and a certain amount of steel in some systems; corrosive effects on steel under certain conditions; rapidity of construction, and allowance for final setting of concrete under certain conditions of weather and of heavy loadings; and comparative cost of different systems.

The weights of hollow-tile floor arches and fireproof materials, in pounds per square foot, are given in the following table:

TABLE XIV.

Weights of Hollow-Tile Floor Arches and Fireproof Materials.

END CONSTRUCTION, FLAT ARCH.

Width of Span Between Beams.	Depth of Arch.	Weight per Square Foot.
5 feet to 6 feet.	8 inches.	27 pounds.
6 " 7 "	9 "	29 "
7 " 8 "	10 "	33 "
8 " 9 "	12 "	38 "

HOLLOW BRICK FOR FLAT ARCHES.

Width of Span Between Beams.	Depth of Arch.	Weight per Square Foot.
3 feet 6 inches to 4 feet 0 inches.	6 inches.	27 pounds.
4 " 0 " 4 " 6 "	7 "	29 "
4 " 6 " 5 " 0 "	8 "	32 "
5 " 6 " 6 " 0 "	9 "	36 "
6 " 0 " 6 " 6 "	10 "	39 "
6 " 6 " 7 " 0 "	12 "	44 "

PARTITIONS.

	Thickness.	Weight per Square Foot.
Hollow Brick (Clay) Partitions	2 inches.	11 pounds.
" " " "	3 "	14 "
" " " "	4 "	15 "
" " " "	5 "	19 "
" " " "	6 "	20 "
" " " "	8 "	27 "
Porous Terra-Cotta Partitions	3 "	16 "
" " " "	4 "	19 "
" " " "	5 "	22 "
" " " "	6 "	23 "
" " " "	8 "	33 "

PARTITIONS — (Concluded).

	Thickness.	Weight per Square Foot.
Porous Terra-Cotta Furring	2 inches.	8 pounds.
" " " Roofing	2 "	12 "
" " " "	3 "	15 "
" " " "	4 "	19 "
" " " Ceiling	2 "	11 "
" " " "	3 "	15 "
" " " "	4 "	19 "

6 inch Segmental Arches, 27 pounds per square foot.
8- " " " 33 " " " "
2-inch Porous Terra-Cotta Partition, 8 pounds per square foot.

The following table shows the safe loads in pounds per square foot uniformly distributed for hollow-tile floor arches.

TABLE XV.

Safe Loads Uniformly Distributed for Hollow-Tile Arches.

Nominal Depth.	Effective Depth. R	Span of Arch in Feet = L					
Inches.	Inches.	3	4	5	6	7	8
6	3.6	336	189	121			
7	4.6	429	242	155			
8	5.6	523	294	188	131		
9	6.6	616	347	222	154	113	
10	7.6	709	399	255	177	130	100
12	9.6	896	504	323	224	165	126

Gross loads in pounds per square foot, *i. e.*, including weight of arch. Safety factor 6.

Nominal Depth.	Effective Depth. R	Weight of Arch per Square Foot.	Span of Arch in Feet = L.					
Inches.	Inches.	Pounds.	3	4	5	6	7	8
6	3.6	27	309	162	94			
7	4.6	29	400	213	126			
8	5.6	32	481	262	156	99		
9	6.6	36	580	311	186	118	77	
10	7.6	39	670	360	216	136	91	61
12	9.6	44	852	460	279	180	121	82

Net loads in pounds per square foot, *i. e.*. excluding weight of arch. Safety factor, 6.

The formula for safe load used in computing the above table is as follows :

$$W = 840 \frac{R}{L^2}$$

in which

W = Safe load per square foot of arch in pounds.
R = Rise or effective depth of arch in inches.
L = Span of arch in feet.

In the following table are given, in pounds per square foot, the weights of various materials used in floor and roof construction:

TABLE XVI.

Weights of Materials in Floor and Roof Construction.

SUBSTANCE.	AVERAGE WEIGHT IN POUNDS PER SQUARE FOOT.
Corrugated galvanized iron, No. 20	$2\frac{1}{2}$
Copper, 16-oz. — Standing seam	$1\frac{1}{4}$
Glass, $\frac{1}{4}$ inch thick	$3\frac{1}{2}$
Cinder-concrete filling, 2-inch, including screeds	12
Plaster, on wood lath (no furring)	6 to 8
Plaster, on metal lath (no furring)	8 to 10
Plaster ceiling, suspended	15 to 20
Roofing felt, 2 layers	$\frac{1}{2}$
Slate, $\frac{1}{8}$ inch thick, 3 inches double lap	$4\frac{1}{2}$
Shingles, $\frac{1}{3}$ to weather	2
Gravel composition roof, 5 ply	9 to 11
Tin, 1 X	
Tiles, $6\frac{1}{4}$ in. × $10\frac{1}{2}$ in. — $5\frac{1}{4}$ in. to weather (plain)	17
Tiles, $10\frac{1}{2}$ in. × $14\frac{1}{2}$ in. — $7\frac{1}{4}$ in. to weather (Spanish)	9
Trusses — Spans under 50 feet	$3\frac{1}{2}$ to $4\frac{1}{2}$
Trusses — Spans 50 to 75 feet	$4\frac{1}{2}$ to $6\frac{1}{2}$
Trusses — Spans 75 to 100 feet	$6\frac{1}{2}$ to 8

PARTITIONS.

Partitions are of terra cotta, wire lath and plaster, and plaster board.

Illustrations of each are given by Plate VI, Figs. 71 to 77. The element of strength does not form a specially important consideration here, as the standard forms are all suitable. The higher the partition the thicker should be the blocks or the heavier the metal frame of the partition. Some of the forms are more sound-proof than others and probably more fireproof, but the use of any one is generally determined by architectural conditions. The terra cotta blocks come in standard sizes given by the table below, which also gives the dead weight per square foot. The constructions around openings in partitions, for the different types of partition, are also shown by the above-mentioned cuts.

Partitions are never as fireproof as the floor system in a building. If a form of construction could be used which would prevent the spread of fire through partitions, the modern office building would probably be in truth absolutely, instead of merely in name,

Plate VI *Types of Fireproof Partition Construction*

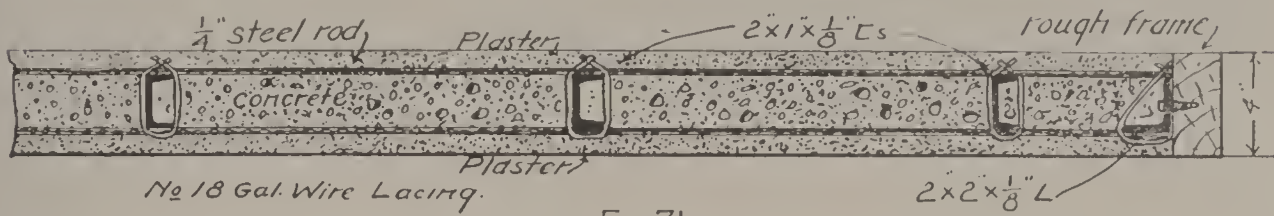


Fig 71.

ROEBLING 4IN. WIRE LATH SOLID PARTITION

Fig.72

ROEBLING 2IN. WIRE LATH SOLID PARTITION.

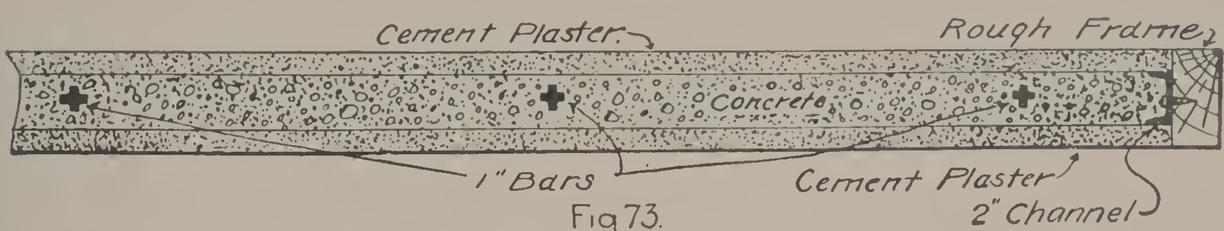
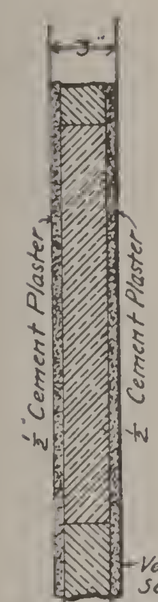


Fig 73.

COLUMBIAN PARTITION.



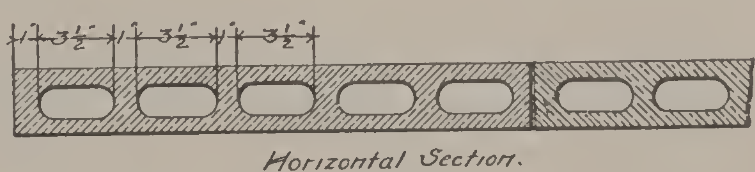


Fig. 74.

PLASTER COMPOSITION BLOCK PARTITION.

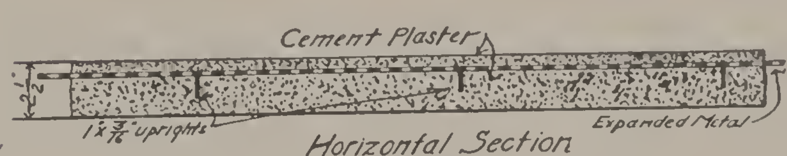


Fig 75.

EXPANDED METAL PARTITION

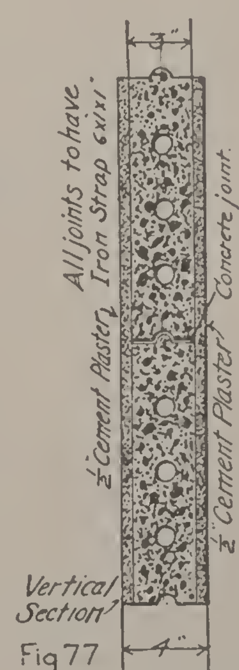


Fig 76.

PLASTER COMPOSITION BLOCK PARTITION.

Fig 77

CEMENT COMPOSITION BLOCK PARTITION.

fireproof. The great cause of the weakness of fire resistance lies not in the partitions themselves so much as in the fact that openings for doors, windows, flues, etc., have to be made in them. The arrangement in a great many buildings makes it necessary, in order to give light in the corridors, to have a line of windows in the partitions between them and the offices. In addition there are the

doors into the corridors, and the doors and sometimes windows in partitions between offices.

As stated under "Building Laws and Specifications," some cities require in buildings of a certain height the use of metal or of fireproof wood for all inside casings and finish, but in the majority of buildings these are not used. Sometimes, also, where plaster and wire lath partitions are used, the plaster does not extend to the floor, and the baseboard has therefore no fireproof protection back of it.

All these features indicate the real elements of weakness in a fireproof partition, and on the extent to which they can be eliminated depends the utility of the partition as a fire barrier. As will be shown later under the paragraphs on tests, there are a number of forms of partition that can be used, which, if without openings and the other features mentioned above, will form effectual barriers. The extent to which fireproof wood and metal overcome the difficulties will be discussed farther on.

Tests of Partitions. Numerous fire and water tests of partitions have been made by the New York Building Department. The partitions were of four general classes:—(1) plaster blocks; (2) blocks of cinder concrete; (3) wire lath plastered with King's Windsor cement; (4) blocks of terra cotta. The partitions were $2\frac{1}{2}$ inches and 3 inches thick. All were exposed to as nearly the same conditions as possible, which were:—a temperature gradually increasing from 500° to 1,700° during a period of one hour, and then a stream of water applied for $2\frac{1}{2}$ minutes. Fire in no case passed through any of the structures; but in the case of most of the plaster block partitions the blocks were calcined slightly in certain places, and the water had washed portions away to a depth of $\frac{1}{2}$ inch to $1\frac{1}{4}$ inches.

The wire lath partitions did not show calcination, but showed to a greater or less extent the effect of the water in the washing away in spots of the browning coat and scratch coat, and, in some instances, in exposure of the lath or metal supports.

The cinder-concrete blocks showed no effect of either fire or water, except that the plaster on the blocks was stripped off.

The terra cotta blocks stood much the same as the concrete, no effect appearing in the partitions themselves, but the plaster being stripped off.

The chief differences, therefore, seemed to appear in the capacities of the various types of partition to withstand the force of water. Those partitions having a harder and less porous structure stood much the best.

From a consideration of the above tests, it will be seen that some forms of partition, under certain conditions of exposure in case of fire, will prove to be more difficult than others to repair, even though they may not entirely fail. Plaster, constituting the finish surface, could not be expected to stand, and does not in a severe fire; the expense, therefore, of this item in the repair would be essentially the same in all forms of partition.

With some of the plaster board partitions in which the blocks were hollow, the calcination and the stream of water broke through the outer shell, leaving the cells exposed. In such cases it would probably be necessary to provide new blocks, as the old ones could not well be repaired. In the solid plaster board blocks the wear, if not more than $\frac{3}{4}$ inch, could probably be repaired by hard plaster, so that, although not being as good as it was originally, the partition, in case of another fire, would still be considered reasonably safe.

The wire lath partitions cannot be considered fireproof until they are plastered. Here, accordingly, the plaster forms an essential feature of the partition; and in case of any considerable portion of this being destroyed and exposing the metal frame, the partition could be repaired by replastering, provided the metal frame had not been injured.

The concrete blocks and the terra cotta blocks in the tests cited above were not injured by the fire and water test; and so, if the results under actual conditions were always as favorable as in these artificial instances, the expense of repairing this form of partition would appear to be less than in the case of the other forms. It should be noted, however, that the partitions tested were without openings, and that openings in a partition weaken its lateral stability. While the block partitions were uninjured, they might not show so favorable results where openings occur, because of the attendant loss of lateral strength. In this respect it is probable that the plaster and wire lath partitions, and those plaster board partitions having metal stiffening, would not be any more liable to

failure with openings than without, because, as constructed, the metal frame is secured at floor and ceiling, and, where openings occur, the frame is also tied longitudinally.

Column Coverings. The particular form of covering to be used is affected by the section of the column. In general, however, this consists of terra cotta blocks, wire lath, and plaster, or a solid block of concrete or plaster. As before stated, the principal source of failure in all forms of covering is their liability to crack off or be knocked off. The more nearly, therefore, the covering can approach a monolith of substantial thickness, the better it will be. If it consists of blocks, these should be bonded or anchored so as to tie the whole together, and should be made with one and preferably two air spaces. If of plaster on wire lath, it should be cement of sufficient thickness; and if of concrete, cast in place, it should form a solid casing without joints and with an air space between it and the steel. In many cases, pipes are run in the column enclosure, so that in such instances the solid monolith is not practicable.

Corrosion of Steel. An important feature in all concrete-steel systems is the effect of the concrete on the steel. Some authorities have held that, on account of its alkaline nature, the presence of Portland cement in concrete is sufficient to prevent any corrosion of the steel. Observations of actual structures, and tests specially conducted, have shown, however, that under certain conditions steel will rust when imbedded in Portland cement concrete, while under certain other conditions it will not rust in such an environment. It has been held by some, for example, that this rusting will not occur unless sulphur is present in the concrete.

Professor Norton of the Massachusetts Institute of Technology has conducted a series of tests to observe the conditions under which steel in concrete will corrode. A number of mixtures of concrete were used, consisting of standard brands of cement and of both cinders and stone. The cinders showed very little sulphur present, and the concretes were distinctly alkaline. The metal imbedded was in the form of steel rods, sheet steel, and expanded metal. The results showed that when neat cement was used no corrosion occurred. It was also demonstrated that when corro-

sion occurred in either the cinder or stone concrete, it was coincident with cracks or voids in the concrete which allowed the moisture and carbon dioxide to penetrate. If the concrete was mixed wet, so as to form a watery cement coating over all the steel, this coating protected the metal even when cracks and voids were present.

Professor Norton announced the further conclusion that when rusting occurred in cinder concrete it was due to the iron oxide or rust in the cinders, which acted as a carrier of the moisture and carbon dioxide, and it was not due to the presence of sulphur. Also, that if cinder concrete was well rammed when wet, and was free from voids, it was about as effective as stone concrete in preventing rust.

His conclusion as regards the part played by rust in itself aiding the further corroding action by assuming the role of carrier for the active agents, shows the importance of having the steel free from rust when it is imbedded in the concrete.

The above observations and conclusions are of the utmost importance as establishing the conditions under which, in both stone and cinder concretes, steel may reasonably be expected not to corrode, and as showing clearly the precautions and methods that should be observed in such construction.

Paints. Paints used for the protection of steel, consist, like all other paints, of a pigment and a vehicle. The pigments used are generally red lead, iron oxide, carbon, and graphite. The vehicle commonly used is linseed oil; and generally this is boiled oil, although raw oil is sometimes used.

Observations covering a period of about four years were made by Mr. Henry B. Seaman, Member of the American Society of Civil Engineers, on various kinds of paint exposed to the locomotive smoke and gases on viaducts over the Manhattan Elevated Railroad in New York City. His report, published in the New York *Evening Post*, concludes that carbon and graphite paints stand such exposure rather better than others, and the carbon paints somewhat better than the graphite. None was entirely efficient. A detailed paper on paints for steel was prepared by Mr. G. M. Lilley, Associate Member of the American Society of Civil Engineers, and was published in *Engineering News*, April 24, 1902.

The value of paints as agents in the prevention of rusting of steel depends much upon the conditions under which the painting is done, the quality of the paint, and the treatment of the metal after painting.

The experiments of Professor Norton, already mentioned, have established that the essential thing is a coating of the steel which will not crack or peel off and is non-porous, and that the steel must be clean. The fact that in many cases paint has been applied over a coating of rust, does not, of course, afford any reason for condemning the use of paint because of its failure in such cases to prevent further corrosion.

If the paint can be applied in such a way as to form for the steel a continuous coating that will not crack, or blister, or peel off, it will probably be a very effective preventative of rust. All paints, however, are more or less porous, and to this extent inefficient.

It is, however, the opinion of the authorities who have given this subject most study, that, while more expensive, a thin coating of Portland cement applied continuously to a clean surface of steel is more effective than paint.

The alkaline character of the cement neutralizes the carbon dioxide which may be present, or which may tend to filter through to the steel. In this regard, therefore, it is probable that a small degree of rust in the steel before it is coated with cement would not be likely to cause further rust, as would be the case if the coating were of ordinary paint, since the carbon dioxide present in the rust would be neutralized by the cement.

FIRE-RESISTING WOODS.

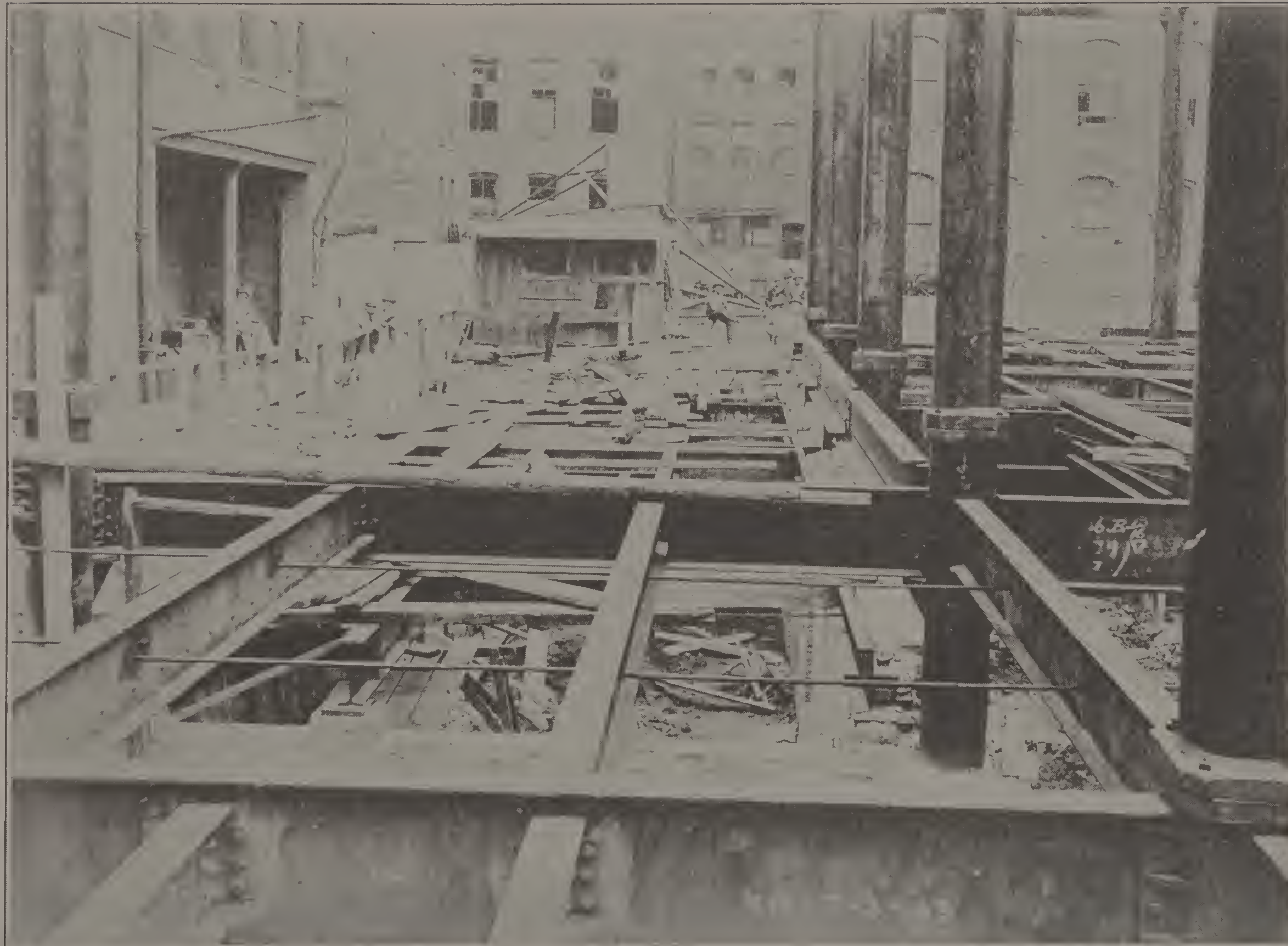
There are several companies who have processes of treating wood to render it fire-resisting. These processes differ materially. None of them renders the wood absolutely fireproof, and tests have conclusively established that all such treated woods will burn if subjected to sufficient heat for a considerable time. Some authorities place this temperature limit at which ignition will occur, as low as 100° above the temperature required to burn untreated wood. Other authorities claim that the period during

which wood will glow after it has been ignited and the flame removed, is as 1 to 10 for the treated and the untreated woods respectively.

The process of treating woods is to impregnate them with certain chemicals which serve to retard the giving off of combustible gases by the wood under heat, and which also, under the action of heat, themselves give off certain other gases that serve to extinguish combustion when started.

It has undoubtedly been demonstrated that treated wood will burn, and that the gases from it are combustible. It is, however, equally well established that treated wood will not ignite as readily as untreated wood; that it requires a higher temperature to maintain its combustion; and that when the source of heat is removed the wood will cease to glow more quickly than untreated wood.

A material has recently been put on the market in England under the name of "Uralite," which, it is claimed, can be worked like wood; and can be used largely in the same way, that is, either solid or as a veneer to form a fireproof covering. The basis of the material is asbestos mixed with whiting. The finished material is made of several thin layers felted together. For a description of this material, see *Engineering*, August 15, 1902.



KENT BUILDING, CHICAGO

Pond & Pond, Architects; E. C. & R. M. Shankland, Engineers

In this ten-story building, cast-iron columns and steel floor-beams are used. Note connection of girders to columns. They rest on a shelf and have a side support. Note that field connections are bolted. Setting of steel work was started May 29, 1903, and finished October 1, 1903.

STEEL CONSTRUCTION.

PART II.

BEAMS AND GIRDERS.

Determination of Loads. The first step in the calculation of a beam or girder is to determine the exact amount of load to be carried, and its distribution. Loads may be uniformly distributed or concentrated, or both in combination. The case of a simple floor or roof beam usually involves only the calculation of the area carried and the load per square foot. The load per square foot is made up of two parts—namely, dead load, or the weight of the construction; and live load, the superimposed load. The latter is generally specified by law, as noted previously under "Building Laws and Specifications."

The calculation of the dead load has to be made in detail to fit each case. In the case of a floor beam this would consist of the arch between the beams, the steel beams and girders, the filling on top of the arch, the wood or other top flooring, the ceiling, and the partitions. These weights cannot be accurately determined until the spacing and size of beams are fixed; so their features have to be assumed at first. The process in general is illustrated by the following case:

Assume a terra cotta arch 8 inches deep, beams spaced about 5 feet center to center, 3 inches of filling and screeds on top of the arch, a $\frac{7}{8}$-inch hemlock under floor, and a $1\frac{1}{8}$-inch oak top floor. The weights then are as follows:

8-in. arch	=	30 lbs.
Steel $= \frac{18}{5} = 3.6$, or say	=	4 "
Filling $= 3 \times 5$	=	15 "
$\frac{7}{8}$-in. floor $= \frac{7}{8} \times 2$, say	=	2 "
$1\frac{1}{8}$-in. top $= 1.125 \times 3.67$, say	=	4 "
Ceiling (no furring)	=	7 "
Partition $= \frac{32 \times 10}{5}$	=	64 "
Total Dead Load		126 lbs.

The calculation of the dead weight per square foot of partitions is made up of the weight of blocks, if of terra cotta, and of the plastering on both sides. If the structure is of wire lath, the weight is that of the framing and plastering. These weights per square foot have already been given in the chapter on Fireproofing.

Only the height of the story is used, as the partition stops at the ceiling. In the above case it is assumed that the partition may go anywhere, and therefore, in some cases, may come directly over a beam, thus being entirely carried by it. If the partitions are in general located so as to come between beams, and no provision is desired for other possible locations, the above partition load might be reduced one-half, as a partition would then be carried by two beams. Or if the partitions came only over girders, the load might be omitted entirely in the calculation of the beams.

In the above total dead load, it should be noted that the allowance for steel does not include the weight of girders. This of course should not be included for the beams. In the calculation of the girders the weight of the girder itself should be added.

The calculation of dead load cannot be absolutely exact, any more than can the determination of the exact amount of live load that will have to be carried. It should always, however, be worked out in detail as above, so that as close an approximation as possible shall be made.

Tables XIV and XVI, of Part I, and Table XVII, Part II, give the weights of different materials and forms of construction, for use in determination of dead loads under different conditions.

The floor arch is assumed to carry all its load vertically to the beams, and the load therefore is the product of the area and the load carried per square foot. This neglect of thrust from the arch is on the safe side as regards the determination of amount of load on the beam.

Distribution of Loads. The load on a girder is generally concentrated at one or more points, and involves the calculation of the reactions from the beams. Girders therefore, as a general thing, are not calculated until after the beams. A girder may also have a uniform load from one side, or from a partition or wall.

TABLE XVII.

Weights of Various Substances and Materials of Construction.

SUBSTANCE.	AVERAGE WEIGHT IN POUNDS PER CUBIC FOOT.	SUBSTANCE.	AVERAGE WEIGHT IN POUNDS PER CUBIC FOOT.
Aluminum	162	Hickory	53
Ash	38 to 47	Iron, cast	450
Asphaltum	62 to 112	Iron, wrought	480
Brass (cast)	490 to 525	Lead, commercial	710
Brick	100 to 150	Limestone	153 to 178
Brickwork	100 to 140	Lime, quick	95
Cement, Portland	80 to 110	Mahogany	35 to 53
Cement, Rosendale	55 to 65	Marble	158 to 180
Cherry	42	Masonry, granite or limestone, dressed	165
Chestnut	41	Masonry, granite or limestone, rubble	154
Clay, Potter's, dry	112 to 143	Masonry, granite or limestone, dry rubble	138
Clay, in dry lumps	65	Masonry, sandstone $\frac{1}{8}$ less than above	
Coal — Anthracite	52 to 60	Mortar, hardened	87 to 112
Coal — Bituminous	47 to 52	Oak, live	60
Coke	23 to 32	Oak, white	47
Concrete — Stone and Portland cement	140	Oak, red	32 to 45
Concrete — Cinders and Portland cement	96	Pine, white	25
Copper, cast	542	Pine, yellow Northern	34
Copper, rolled	555	Pine, yellow Southern	45
Cypress	64	Poplar	29
Earth — Common loam, dry and loose	72 to 80	Platinum	1,342
Earth — Common loam, dry and rammed	90 to 100	Quartz	165
Earth — Common loam, soft-flowing mud	110 to 120	Sand	90 to 130
Elm	35	Snow, freshly fallen	5 to 12
Gneiss, common	168	Snow, moist compacted	15 to 50
Gneiss, in loose piles	96	Slate	175
Gold, cast pure or 24 karat	1,204	Spruce	25
Gold, pure-hammered	1,217	Steel	490
Granite	160 to 178	Sycamore	37
Gravel	90 to 130	Tar	62
Hemlock	25	Terra Cotta	106
		Terra Cotta masonry	112
		Tin, cast	459

NOTE. Where weights of wood are given above they are for perfectly dry wood. Green timbers weigh from one-fifth to one-half more than dry, ordinary building timbers, one-sixth more than dry.

thus bringing sometimes very unsymmetrical loading. Openings also affect the distribution of loading on a beam or girder.

Stairs should be figured as fully loaded with the assumed live load and the dead weight of their own construction, and as being supported by the girder on which they rest. In the case of very heavy live loads, as in warehouses, the customary live load in office buildings could be used in determining the load for stairs.

If the framing plan is drawn accurately to scale, the position of concentrated loads can be determined by scaling. In the case of short girders with heavy loads, however, a slight error in determining the position of loads would appreciably affect the result; hence it is necessary to exercise caution in scaling the position, to avoid any chance of great variation from true measurement.

Beams and girders carrying elevator machinery should have the loads and their position determined with special care. To this end the layout of the company installing the machinery should always be used in final calculation. This layout gives the loads at the different points; and therefore the exact position on the supporting beams, and the reaction on the girders, can be determined. As elevators are liable to cause a shock in sudden starting and stopping, it is customary to multiply the total load by two to allow for this shock.

In the calculation of the girder the laws of some cities allow a reduction amounting to a certain percentage of the live load, on the assumption that the whole area adjacent to a girder is not likely to be loaded to its maximum at the same time. This, however, should not be done in warehouses, nor where on the other hand the assumed loads are very light; and in any case it should be done with discretion.

Lintels. The size and character of lintel beams depends (1) on load to be carried, (2) on arrangement of openings over beams, (3) on practical considerations of construction.

If the wall is solid above the opening for a height greater than the span of the opening, the masonry, if of brick, will arch to some extent and thus relieve the lintel of a portion of the load. Practice varies in the proportion of load assumed to be carried. It is good practice to consider the weight of a triangular section of wall, of height equal to the span, as carried by the lintel. If

there is only a small pier under the ends of such a lintel, however, this arch effect should not be considered, but the full load of masonry provided for. In very wide openings, also, the full load should be calculated on the lintel. The basis for assumption of arching effect is that brickwork can be corbeled out at an angle of about 60°, and support safely its own weight after final set in the cement has taken place. This assumption should not be made where the center of gravity of such mass of masonry will fall outside the supporting base. The figures below will illustrate this principle.

Another assumption sometimes made is, that the wall spanning the opening is capable, as a beam, of carrying a certain portion of the load, and that the lintel need be calculated only for the additional weight. This is necessarily dependent on the tensile strength of the mortar joints, which, although being considerable in an old wall, would be very slight in a new wall; and for new work, therefore, this assumption should not be made.

The arrangement of openings above the lintels often makes it necessary to provide for the full load of wall, because this load is carried in the direct line of piers to the lintels. Such cases are illustrated by the figures below.

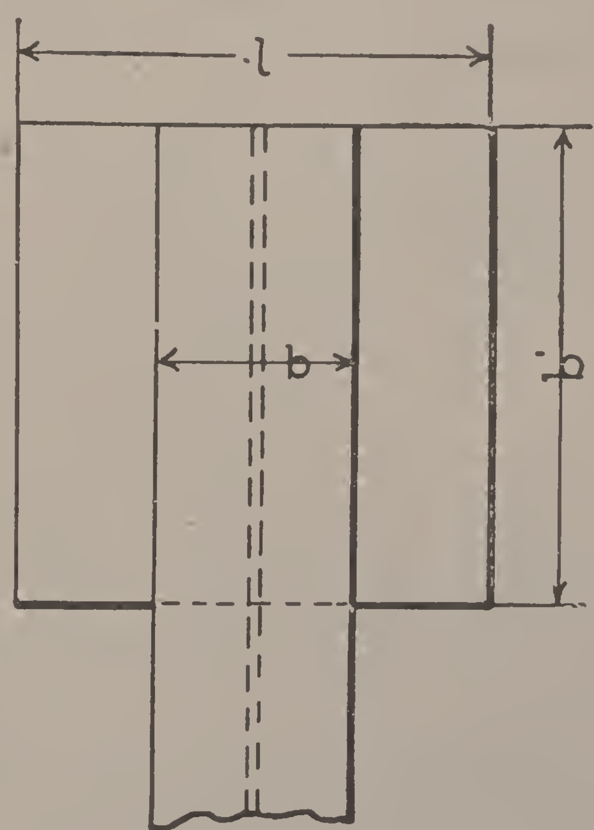


Fig 78

The particular form of lintel will depend not only on the load, but on the way in which the metal must be distributed in order to carry the load. A very thick wall may necessitate a number of beams or other shapes to provide necessary width on which to lay the brickwork. If the stone or terra cotta facing has to be supported, this also necessitates special shapes to meet the requirements. Moreover, if floor loads are to be carried, the size and shape will be largely fixed by this further condition. A lintel may, therefore,

consist of a number of different shapes of different sizes. The problems below illustrate types of condition ordinarily met with.

Beam Plates. Beams and girders carrying ordinary loads, usually have plates under the ends resting on the walls, in order properly to distribute the load on the masonry.

The method of determining the proper size and thickness of such plates is as follows:

In Fig. 78, l = dimensions of plate in inches transverse to web of beam;
b' = dimension of plate in inches in direction of web of beam;
b = width of flange of beam.

The plate should cause the load to be uniformly distributed on the masonry over its whole area.

If R = the reaction at wall end,

then $\frac{\mathrm{R}}{b'\,l}$ = the load per square inch on masonry.

The portion of the plate not covered by the flange of beam is in the condition of a beam fixed at one end and free at the other. The formula for the moment, therefore, is:

$$\mathrm{M} = \tfrac{1}{2}\,p\,\mathrm{L}^2$$

$$p = \frac{\mathrm{R}}{b'\,l}, \text{ and } \mathrm{L} = \frac{l-b}{2}$$

$$\text{therefore } \mathrm{M} = \tfrac{1}{2} \times \frac{\mathrm{R}}{b'\,l} \times \left(\frac{l-b}{2}\right)^2$$

considering a strip 1 inch in direction of web of beam; but from the formula for beams,

$$\mathrm{M} = \frac{f\,\mathrm{I}}{y}\text{; if, therefore, } t = \text{thickness of plate,}$$

$$= \tfrac{1}{6}\,f\,b\,t^2\text{; then, since } y = \frac{t}{2},$$

therefore $\dfrac{6\ M}{f\ b} = t^2 = \frac{1}{2} \times \dfrac{R}{b'\ l} \times \dfrac{(l-b)}{4} \times \dfrac{6}{f}$, since b = 1,

which reduces to $t^2 = \dfrac{3}{4} \quad \dfrac{R(l-b)^2}{b'\ l\ f}$

$$t = .866\ (l-b) \sqrt{\frac{R}{b'\ l\ f}}$$

For steel plates, $f = 16{,}000$
For cast iron $f = 2{,}500$.

The safe bearing on masonry has been specified in the chapter on Building Laws and Specifications.

If two or more beams spaced close together were used, then b in the above formulæ would be the extreme distance between flanges of outside beams.

Anchors. Beams resting on brick walls are anchored to these walls. Some of the more common forms of anchors are shown by Figs. 79 to 86.

Separators. When two or more beams are used together to form a girder, they are bolted up with separators. These separators are either bolts running through spool shaped castings of the required length to fit between the webs of beams, or plate-shaped castings made to fit accurately the outlines of the beams and having width equal to the space between webs of beams. The object of these separators is two-fold; (1) to prevent lateral deflection of the beams under the loading; (2) to distribute the loads equally between the beams when the loads are not symmetrical on the two beams, and to cause the beams to deflect equally. The latter function is by far the more important one, and for this purpose the second form of separator is the only one that should be used. Beams over 12 inches deep have, as a general thing, two horizontal lines of separators; beams under 12 inches, one horizontal line.

Figs. 87 to 89 illustrate the different types of separator.

Calculations. *To find the actual fibre stress on a given beam supporting known loads:*

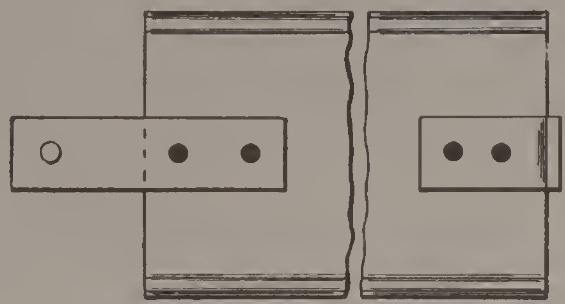


Fig. 79

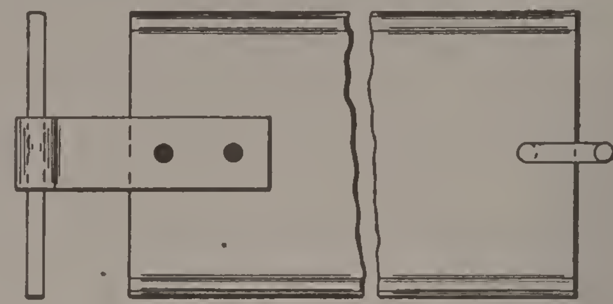


Fig. 80

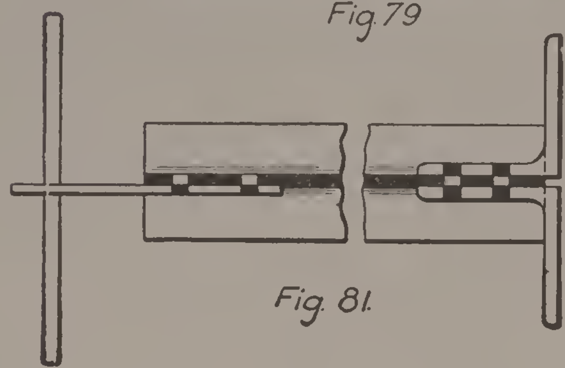


Fig. 81

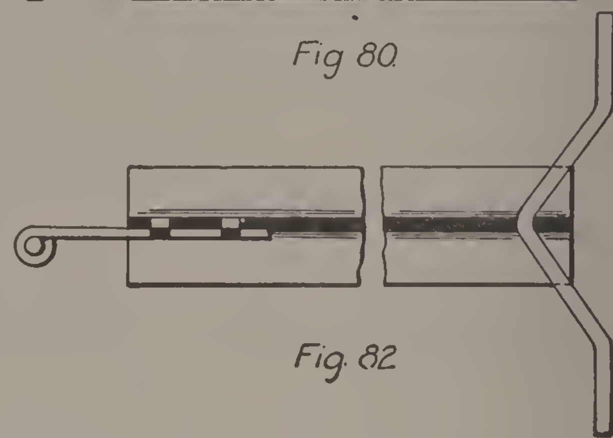


Fig. 82

Fig. 83

Fig. 84

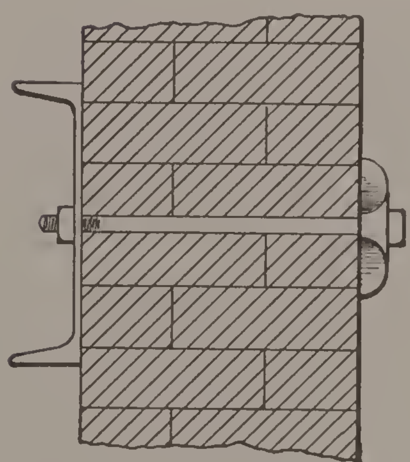


Fig. 85

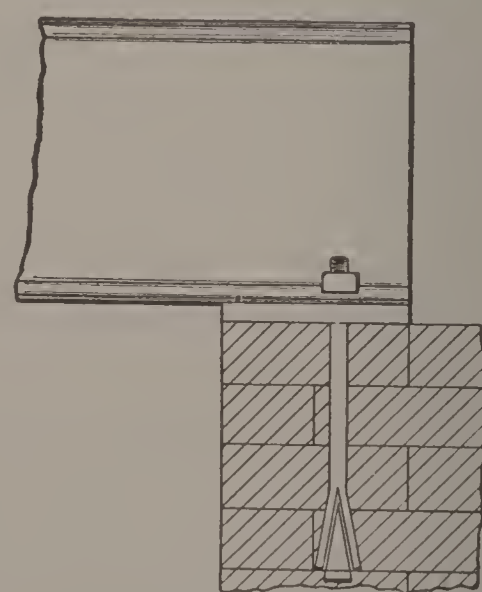


Fig. 86

Data required:

1. Length of span of beam, center to center.
2. Size and weight per foot of beam.
3. The amount and character of load on the beam.

Operations:

1. Find from the tables in Cambria the moment of inertia of the beam.

2. Figure the bending moment due to all the concentrated loads, and the uniform load in inch-pounds.

3. Apply formula $f = \frac{M\,y}{I}$.

Substituting the values obtained above we find the value of f.

NOTE. Since we know the size of beam, the value of y is one-half the depth of beam.

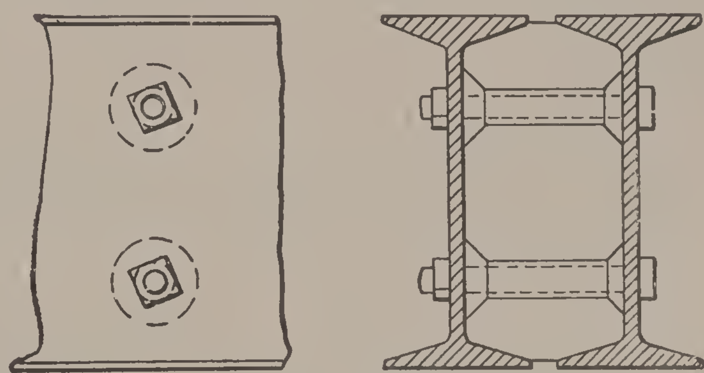


Fig. 87

CAST IRON SPOOL SEPARATORS

A more direct method would be to find the value of S (see Cambria) and dividing M by S which would give the required fibre stress.

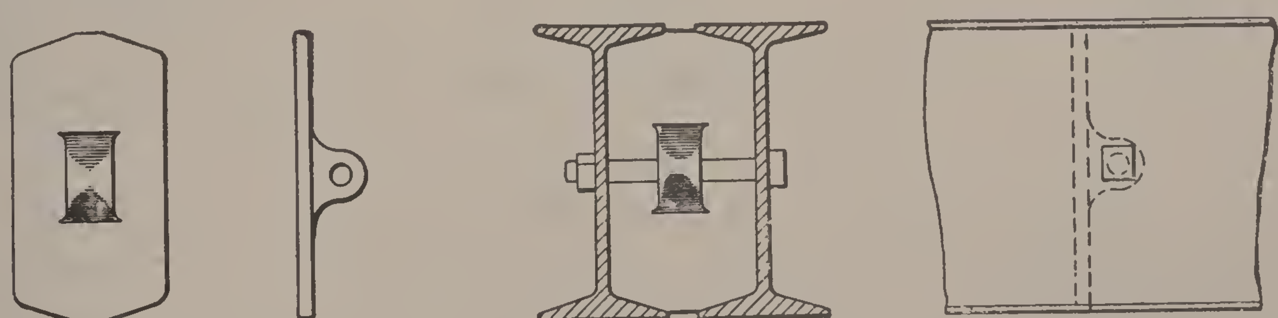


Fig. 88

STANDARD CAST IRON SEPARATOR WITH ONE BOLT

To find what load, uniformly distributed, will be carried by a given beam at a given fibre stress.

Data required:

1. Length of span, center of bearings.
2. Allowed fibre stress.
3. Size and weight per foot of beam.

Operations:

1. Find from the tables the moment of inertia of the given beam.

2. Find the value of the beam in bending-moment, inch-pounds, from the formula $M = \frac{f\,I}{y}$

3. Find the value of the beam in bending-moment foot-pounds by dividing the result obtained under operation 2 by 12.

4. Find the value of W in the formula

$$W = \frac{8\ M}{l},$$

in which W = the total load in pounds uniformly distributed which the beam will support:

M = the bending moment in foot-pounds; and
l = length of span in feet.

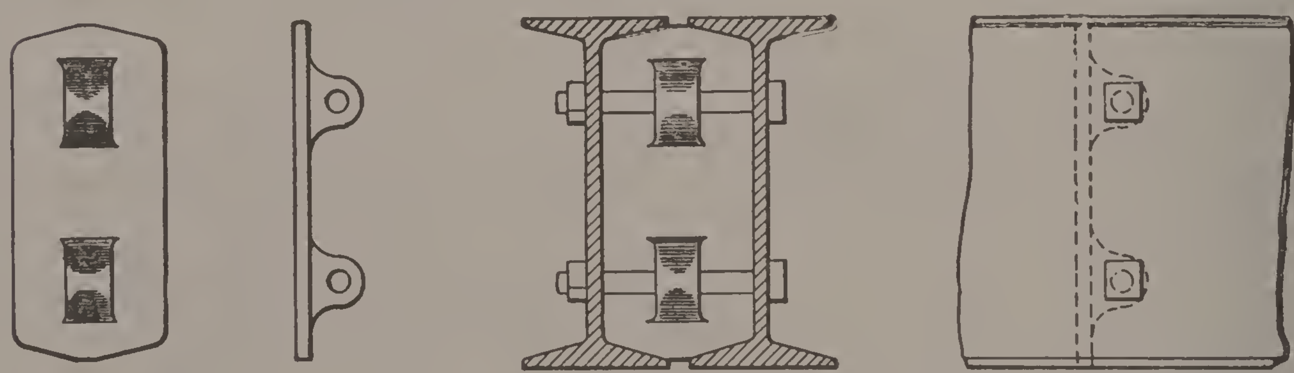


Fig. 89.
STANDARD CAST IRON SEPARATOR WITH TWO BOLTS.

To find the size of beam required to carry a system of known loads at a given fibre stress.

Data required:

1. Length of span, center to center.
2. Allowable fibre stress.
3. The amount and character of load on the beam.

Operations:

1. Figure the bending moment in inch pounds due to all the concentrated loads, and the uniform load.

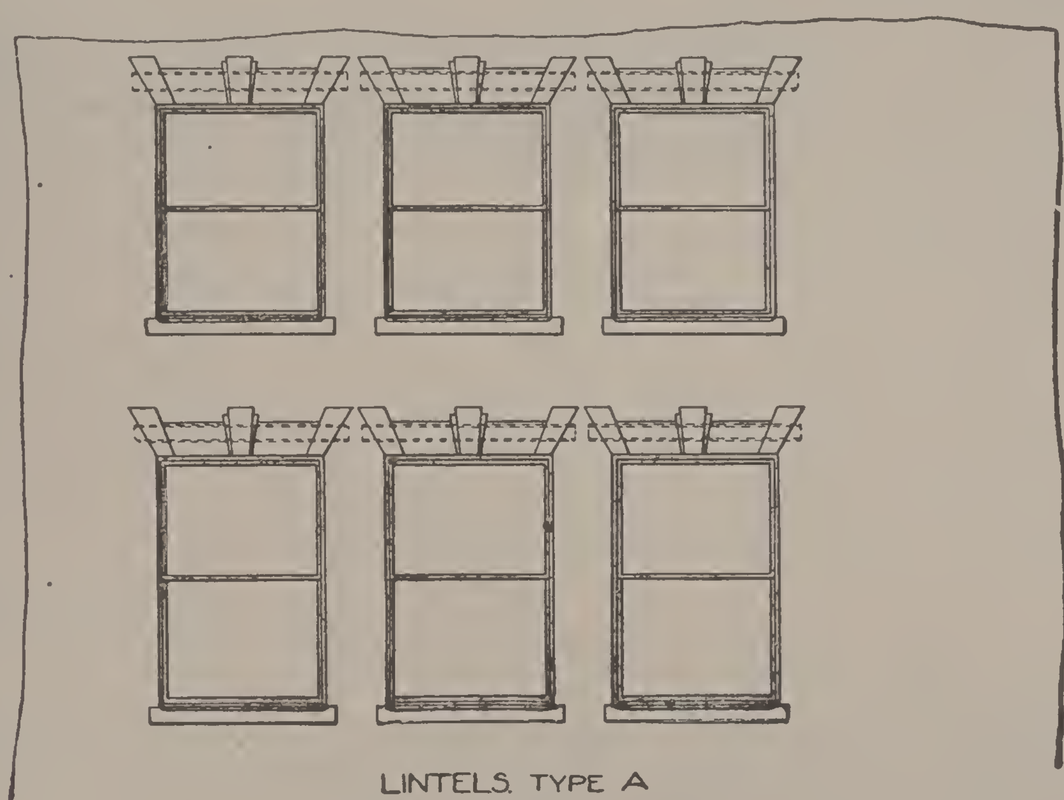
2. Divide the bending moment in inch pounds by the specified fibre stress, and the result will be the required section modulus, S.

3. Select from Cambria a beam having the required value of S.

NOTE. Due attention in selecting the beam must be given to lateral and vertical deflection as previously noted, or to a proper reduction of the specified fibre stress to allow for these considerations.

PROBLEMS FOR PRACTICE.

1. Given a 15-inch 60-lb. beam on a span, center to center of bearings, of 22 feet 6 inches. Required the safe load uniformly distributed at a fibre stress of 16,000 lbs. per square inch.



LINTELS. TYPE A

Fig. 90

Solve (*a*), by the methods given above;

(*b*), by use of coefficient of strength given in table of Properties by the formula $M = \frac{C}{8}$.

2. Find from the table of Safe Loads the total load which a 6-inch 12.25-lb. beam will carry on an effective span of 15 feet, without exceeding the limits of deflection for plastered ceiling; allowable fibre strain 16,000 lbs. per square inch.

What would be the safe load in the above problem if the allowable fibre strain were 10,000 lbs. per square inch?

In the following problems, solve,

(*a*) by use of tables of Safe Loads, and

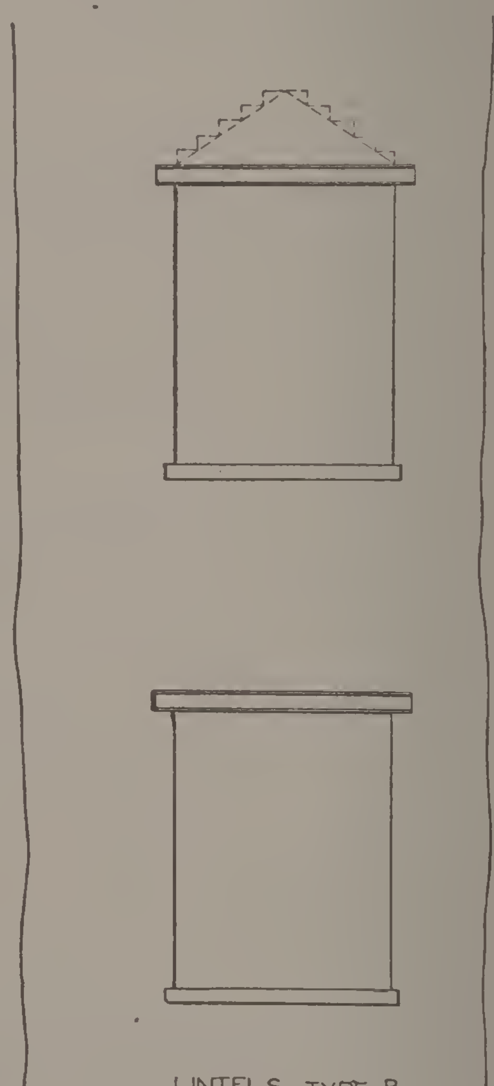
(*b*) by formula $M = \frac{f\,I}{y}$, and use of table of Properties.

3. Find the greatest safe load in pounds uniformly distributed that will be sustained by a 10-inch 35-lb. I beam having a clear span of 10 feet 3 inches and an effective span of 11 feet 3 inches, the allowed stress in extreme fibre being 12,500.

4. The moment of the forces in foot-pounds acting on a beam of undetermined size is 108,000. What size of beam will be required if a stress of 16,000 pounds per square inch is allowed in extreme fibre?

5. What load uniformly distributed will a 15-inch 42-lb. I beam support per linear foot, if the span, center to center of bearings, is 10 feet 4 inches, and the allowed stress in extreme fibre is 14,500 pounds per square inch?

6. What weight of wall will a 12-inch 31.5-lb. I beam 18 feet long between center of bearings carry, no transverse support for wall? Allowable fibre strain, 16,000 lbs. per square inch.



LINTELS TYPE B.
Fig 91

7. An office building has columns spaced 15 feet on center in both directions. Give in detail the estimates of dead load for the following constructions. Live load in each case 100 lbs. per square foot.

(*a*) Beams spaced 5 feet center to center, 8-inch terra cotta arch of end construction, 2-inch wood screeds and cinder concrete filling, $\frac{7}{8}$-inch under floor, and $\frac{7}{8}$-inch maple top floor.

(*b*) Same conditions, except 8-inch terra cotta arch of side construction.

(*c*) Same spacing of beams, with expanded-metal arch, type 8.

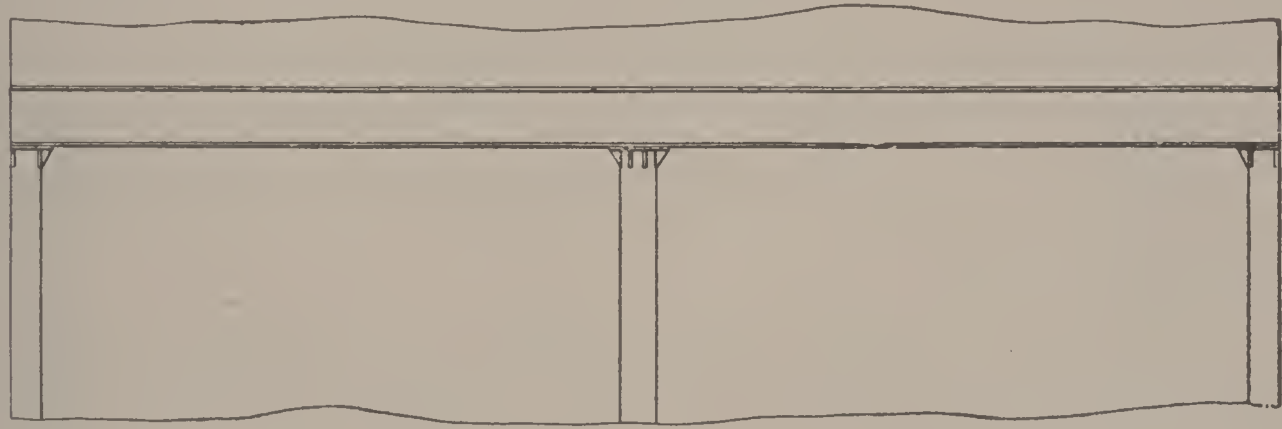
(*d*) Same conditions as above, but expanded-metal arch, type 3, with suspended ceiling.

(*e*) Beams spaced 7 feet 6 inches center to center. Columbian system, type 2, stone concrete.

NOTE. In all above cases, partitions are of 3-inch terra cotta blocks, and come only over girders. Clear story height = 10 feet. Give loads for both beams and girders.

8. Required a lintel over opening shown by Fig. 92. Clear span 15 feet, wall 16 inches thick and 50 feet high. No floor or any load carried by wall.

In this type of opening, the narrow piers or columns under the lintels make it necessary to figure the full load of wall, as otherwise the narrow base supporting the heavy overhanging mass of masonry would cause at the piers a thrust that would necessitate continuous tie rods. The full load, therefore, would be $50 \times 15 \times 1.33 \times 115 = 115{,}000$ lbs. The effective span of lintel is 16 feet; the capacity of two 18-inch 55-lb. I beams for this span is 117,800 lbs., and these would, therefore, be the required sections.



LINTELS TYPE C.
Fig 92

Required the size of lintel of type B, Fig. 91. Span between centers of bearings, 7 feet. Wall 20 inches thick. Floor load 200 lbs. per square foot. Columns spaced 15 feet from wall.

In this case the piers at side of opening are sufficiently heavy for us to consider the wall over opening as arching, as shown by dotted lines.

$$\text{Floor load} = 200 \times 7.5 \times 7 \qquad = 10{,}500 \text{ lbs.}$$
$$\text{Wall load} = 7 \times 3.5 \times 1.67 \times 115 = 4{,}697 \text{ lbs.}$$

The full floor load should be provided for. The wall load is not a uniformly distributed load, and moment should be calculated by assuming load between center and end of girder as acting $\frac{1}{3}$ the way from the center of the girder.

$$\text{M of floor load} = \tfrac{1}{8} \times 200 \times 7.5 \times 7 \times 7 \times 12 \qquad = 110{,}200 \text{ inch-pounds.}$$
$$\text{M of wall load} = \frac{7 \times 3.5 \times 1.67 \times 115}{2} \times 2.33 \times 12 = \underline{65{,}500} \text{ `` ``}$$
$$175{,}700 \text{ `` ``}$$

The moment in foot-pounds of wall load can be obtained also by the use of the formula $M = \frac{p\,l^3}{12}$, where p is the weight of a square foot of the masonry of the given thickness, and l the span.

If the allowable fibre strain is 16,000, this gives a necessary section modulus of 11.0.

Two 7-inch 9.75-lb. I beams have a total section modulus of 12.0, and would, therefore, be sufficient.

NOTE. In this calculation the strength of the angle riveted to the channel is not considered in the capacity of lintel.

10. What size of beam will be required to span 19 feet center to center of bearings, and support a uniform load of 1,200 lbs. per linear foot, together with two concentrated loads of 5,000 pounds each? One concentrated load to be applied 7 feet from the left-hand support and the other 8 feet 9 inches from the left-hand support. The allowed fibre stress is 9,000 pounds per square inch.

11. Find the actual stress in extreme fibre of a 12-inch 31.5-lb. I beam spanning 12 feet 6 inches center to center of bearings, and supporting a uniformly distributed load of 23,500 pounds, and one concentrated load of 7,500 pounds placed 4 feet 9 inches from left-hand support.

12. What will be the most economical arrangement of floor beams and girders for carrying a load of 175 pounds per square foot, including weight of floor? Assume floor to be of expanded metal, fireproof construction, and beams spaced not to exceed 6 feet. Under side of floor to carry a plastered ceiling.

13. What size and weight of beam, 23 feet long in the clear between walls, will be required to carry safely a uniformly distributed load of 14 tons, including the weight of beam?

14. What load uniformly distributed, including its own weight, will a 12-inch I beam, 31.5 pounds per foot, carry for a clear span of 23 feet 6 inches, without deflecting sufficiently to endanger a plastered ceiling? Beams rest 12 inches on walls at each end.

15. Calculate by use of Cambria book the moment of inertia about the neutral axis perpendicular to web at center of a 12-inch 31.5-lb. beam.

16. Given a girder loaded as follows: Effective span 28 feet; center load 4,000 lbs.; and a load, 7 feet each side of cen-

ter, of 3,000 lbs. Required the size of beam such that the deflection will not exceed plaster limits.

17. Given a warehouse 180 feet by 80 feet inside of walls. Columns spaced 18 feet longitudinally and 16 feet transversely. Total load per square foot 300 lbs. Required the necessary sizes of beams and girders.

18. In the above warehouse, what changes in spacing of columns longitudinally could be made to give more practicable sections of beams and girders, and what sizes could then be used?

19. Given a girder loaded as shown by Fig. 93. Allowable fibre stress, 16,600 lbs. per square inch. Required:

(*a*) The size of single beam girder.

(*b*) The size of single beam or channel to carry end of girder framing into lintel.

(*c*) The size of double beam girder.

(*d*) The size of double beam or channel lintel.

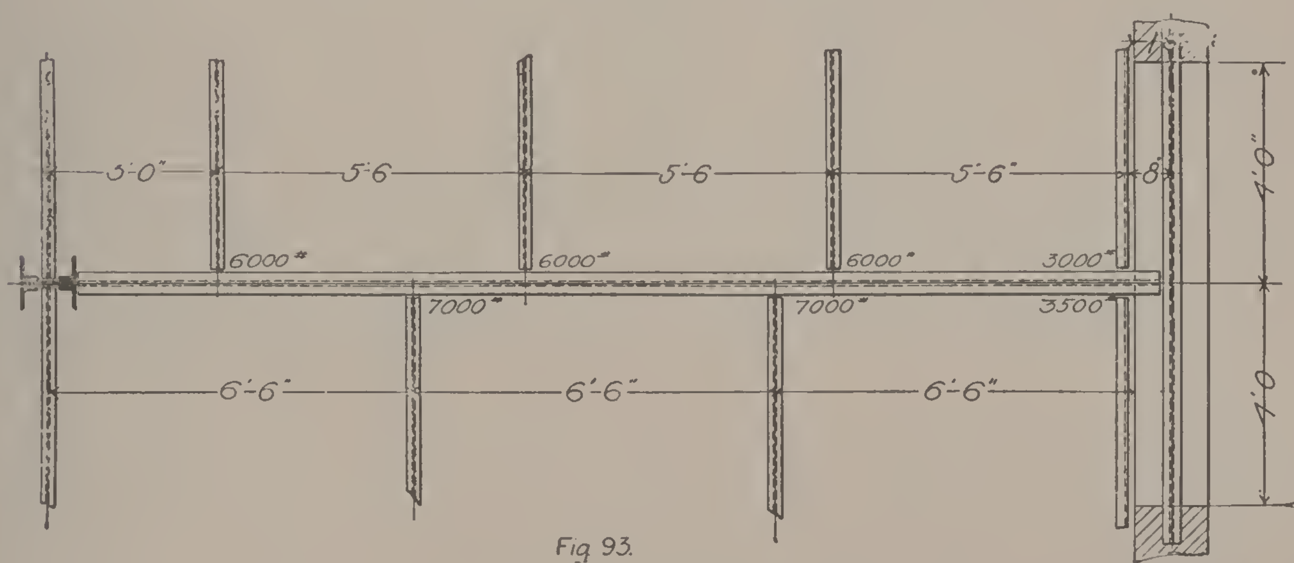


Fig. 93.

20. Given a system of overhead beams for an elevator as shown by Fig. 94. Required the size of beams Nos. 1, 2 and 3. Make allowance for shock as previously stated, and observe that when two beams are used together as a girder they must be of the same depth. Allowable fibre stress 15,000 lbs. per square inch.

In all the above problems, unless otherwise noted, use $f =$ 1,600 pounds per square inch.

COLUMNS.

A column ordinarily has to carry only vertical loads. There are conditions in which it has to resist lateral forces, but these will

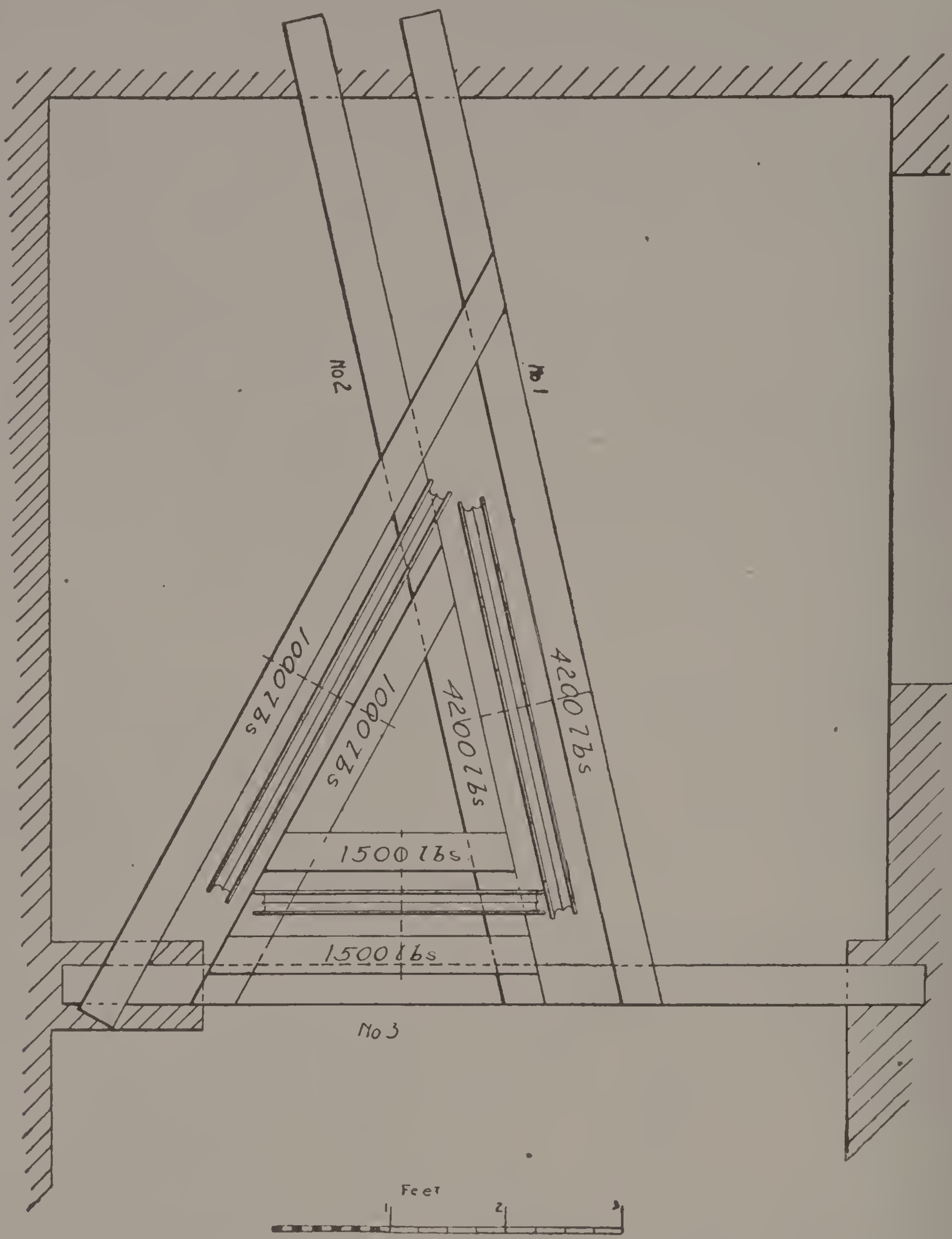


Fig. 94.

be taken up under the heads of "High Buildings" and "Mill Buildings."

Shapes Used. A column may be made of any of the structural shapes that are rolled, or of any combination of them which it is practicable to connect together. In practice, however, there are certain combinations which are commonly used to the exclusion of others. Beams, channels, angles, tees, and zees are all used singly at times, as columns. The more common combination of shapes are shown in Plate I of Part I.

The component parts of these columns will be evident in most cases, from an inspection of the figures. The white spaces between the black lines indicating the different shapes do not represent actual spaces; this is a conventional form to more clearly show the shapes of which the column is composed.

Fig. 5 is a two-angle and a four-angle column. Adjacent legs of the angles are riveted together as indicated. Sometimes plates are riveted between the angles to increase the area of the column or to make simple connections.

Fig. 6 is a four-angle column to which the angles are connected by lattice bars, which come in the position shown by the light line, and run diagonally from side to side of the column for its entire length.

In Fig. 7 a continuous plate is substituted for the lattice bars.

Fig. 8 is a similar column in which one or more plates are added to the outstanding legs, on each side, to increase the area of the column.

Fig. 9 represents a column composed of two channels connected by lattice bars, riveted to the flanges.

In Fig. 10 continuous plates are substituted for the lattice bars.

Fig. 11 is a column similar to Fig. 10, but shows plates riveted to the webs of the channels to stiffen them and to increase the area of the column; these plates have to be riveted before the flange plates are put on.

Fig. 12 is a column of similar shape, but instead of the channels, angles riveted to plates are used. This has the disadvantage, common to Fig. 11, of four extra lines of rivets as compared with Fig. 10. A heavier section can be made, however, than would be possible with any of the channel sections, and a better riveted connection can be made through the flange angle than through the flanges of the channels.

Fig. 13 is known as a "Grey column," and is a patented section. The unshaded lines between the angles represent tie plates which occur about 2 feet 6 inches apart from top to bottom, and serve to connect the angles to each other.

Figs. 14 and 15 are similar to Figs. 9 and 10, the channels being simply turned in instead of out; this is of advantage sometimes in making connections or when a plain face is desired.

Fig. 16 is called a "Larimer column," and is also a patented section. It consists of two I beams bent in the form shown and riveted together through a special shaped filler, shown unshaded. This column has the same advantage as the Grey column, that it gives a flange on all four sides to make connections with. Neither column is very generally used, however, and when used they are subject to a small royalty charge.

Fig. 17 is a modification of Fig. 8, in which channels are used instead of plates. This gives more simple connections of beams, especially where the beams frame eccentrically with regard to the axis of the column. This section also gives a larger radius of gyration, and has many of the advantages of the Z-bar column shown by Fig. 23, although having four extra lines of rivets.

Fig. 18 is a column having four Z bars connected by tie plates spaced about 3 feet apart, and which are indicated by the unshaded lines.

Fig. 19 is similar except a continuous plate is substituted for the interior tie plates.

Fig. 20 is a section intended to give the form of Fig. 17. The rivets through the beam flanges are objectionable, however, except for light loads and short lengths.

Fig. 21 is a modification of Fig. 19, in order to increase the area.

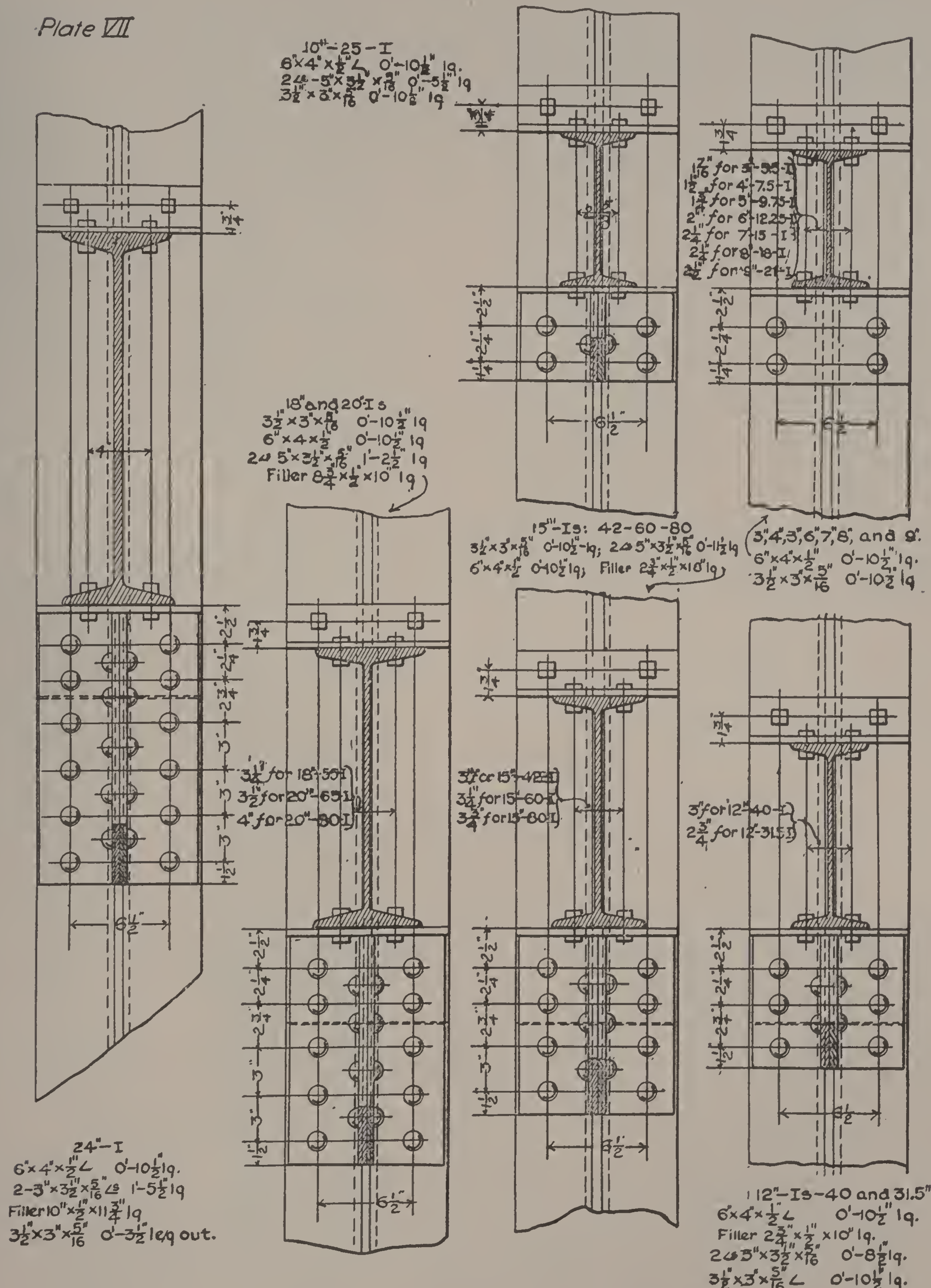
Fig. 22 is a modification of the usual form of Z-bar column shown by Fig. 23. This gives increased area and a greater spread between the outstanding flanges of the Z bars, which is of advantage sometimes in making connections.

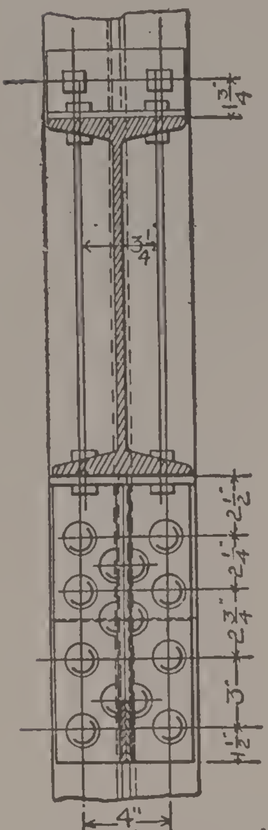
Fig. 23 is the very generally used Z-bar column. This section has its metal so distributed as to give a high radius of gyration, and its shape makes connections simple. Z bars cost about $\frac{1}{10}$ of a cent per pound more than other shapes, and it is not possible, generally, to get so prompt delivery.

Fig. 24 shows the usual method of increasing the area of a Z-bar column by adding plates to the flanges.

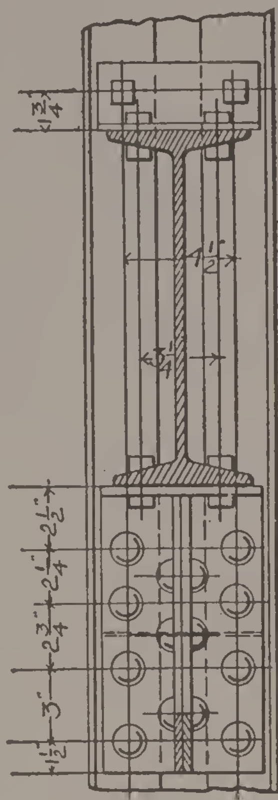
Effect of Connections. In order to design a column intelligently, it is necessary to know in every case how the members that are to carry the load to the column are to be connected to it. Types of connection are illustrated by Plates VII and VIII, Figs. 95 to 105.

Plate VII

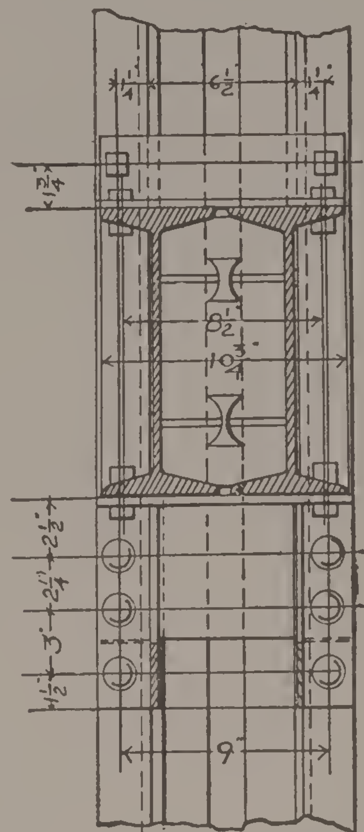




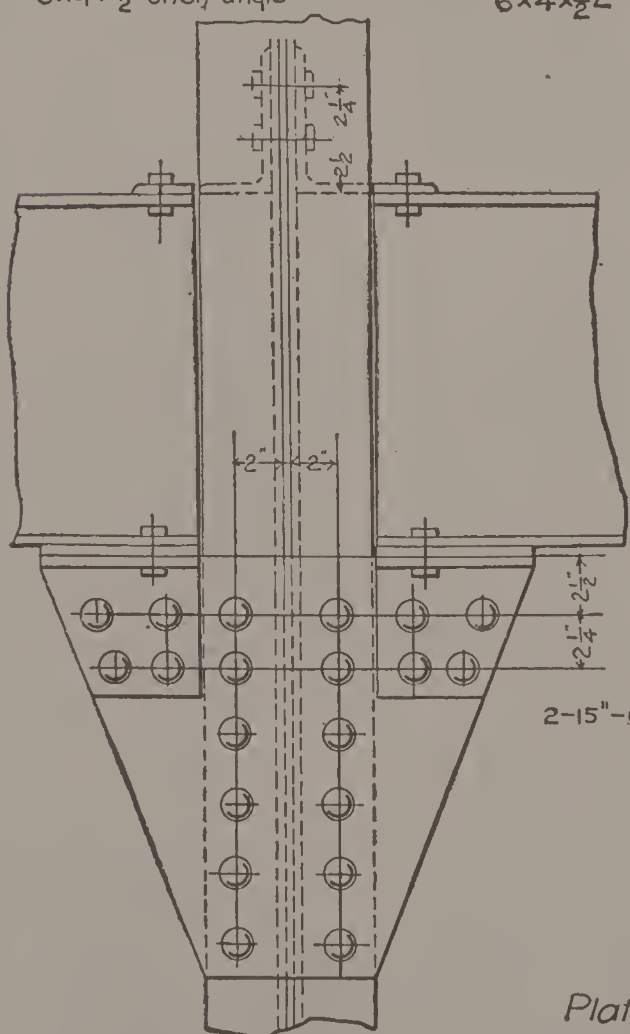
Web 8"×$\frac{1}{4}$"; 4∠s 3"×2$\frac{1}{2}$"×$\frac{1}{4}$" 3 leg out
15"-60-I — 2∠s 3$\frac{1}{2}$"×3"×$\frac{5}{16}$" 0'-11$\frac{1}{2}$" lg
3$\frac{1}{2}$"×3"×$\frac{5}{16}$" cap ∠ — Filler 5$\frac{3}{4}$"×$\frac{1}{2}$"×6$\frac{1}{2}$" lg
6"×4"×$\frac{1}{2}$" shelf angle



Web 8"×$\frac{1}{4}$", 4∠s 3"×2$\frac{1}{2}$"×$\frac{1}{4}$" 3" leg in.
15"-60-I, 2∠s 3$\frac{1}{2}$"×3"×$\frac{5}{16}$" 0'-11$\frac{1}{2}$" lg.
3$\frac{1}{2}$"×3"×$\frac{5}{16}$" ∠ 0'-7" lg. Filler 5$\frac{3}{4}$"×$\frac{1}{2}$"×7 lg
6"×4×$\frac{1}{2}$ ∠ 0'-7 lg.



Web 6$\frac{1}{2}$"×$\frac{1}{4}$"; 4∠s 3"×2$\frac{1}{2}$"×$\frac{1}{4}$"; 2[s 8"-11.25
2-12"-31.5 Is; 3$\frac{1}{2}$"×3"×$\frac{5}{16}$" ∠ 10$\frac{3}{4}$" lg. Filler 2$\frac{3}{4}$"×$\frac{1}{2}$"×3 lg
6"×4"×$\frac{1}{2}$ ∠ 0'-11" lg. 2∠s 3$\frac{1}{2}$"×3"×$\frac{5}{16}$" 0'-8$\frac{1}{2}$"



2-15"-60-Is. 6$\frac{3}{4}$" c.to c.

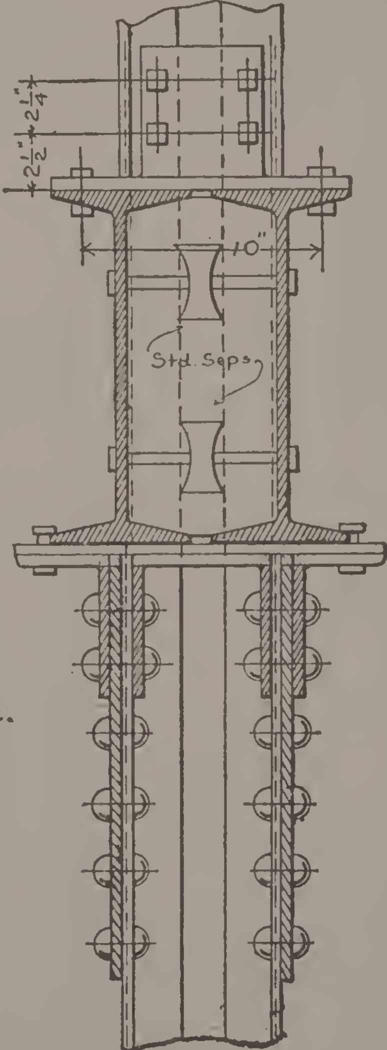


Plate VIII.

There is hardly ever a case in which the loads on a column can be exactly balanced so that the center of gravity of the loads will coincide with the axis of the column. Practically, also, the beams on one side may receive their full load while those on the other side are only partially loaded. The effects of eccentricity

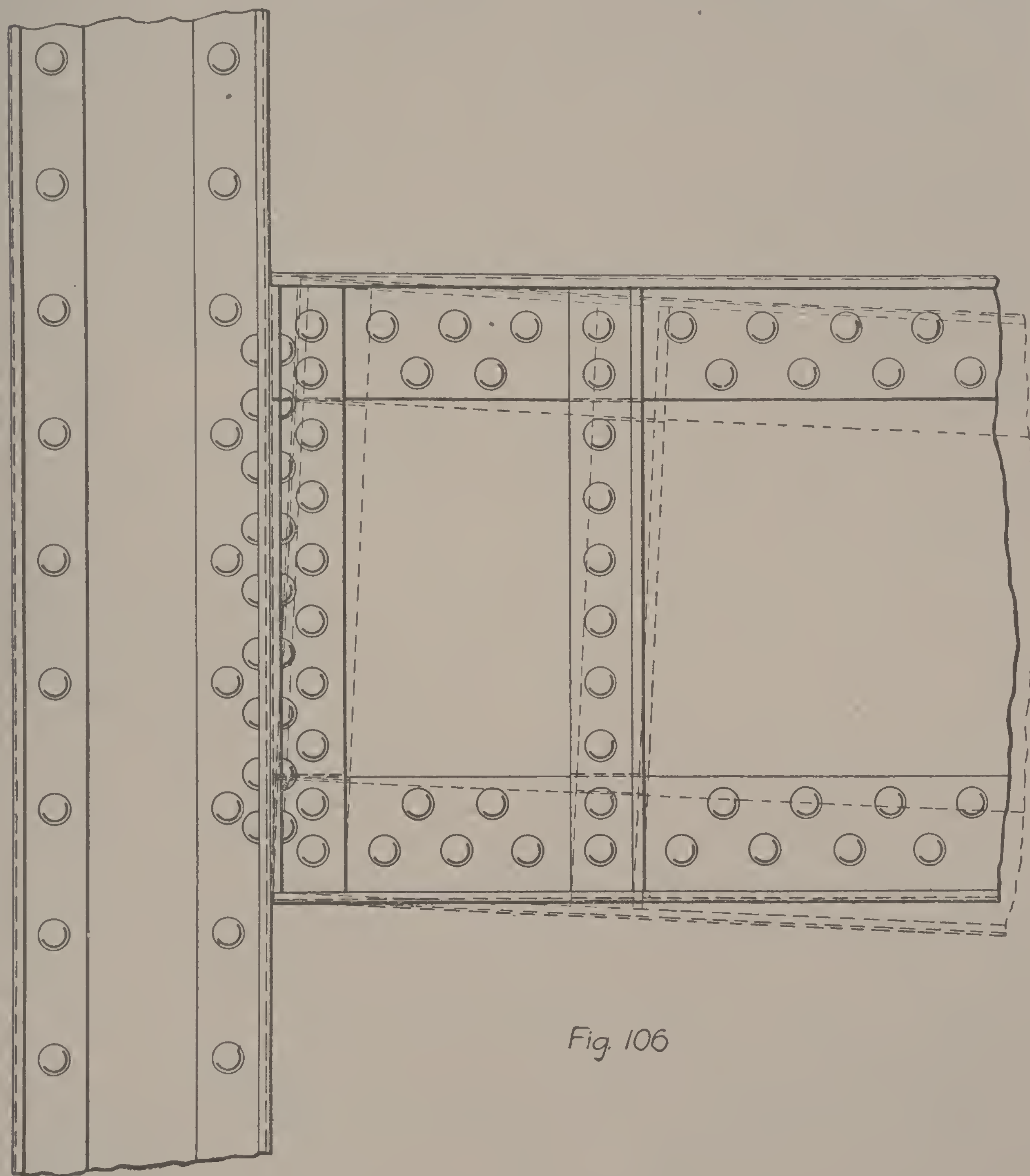


Fig. 106

of loading are very apparent in tests of the carrying capacity of columns; and, where practicable, a column section should be chosen which will admit of connections bringing the loads as near to the axis of column as possible. If the beams frame symmetrically about the axis of the column and are almost equally loaded, it is not generally necessary, in calculation, to consider the effect of

eccentricity. In cases, however, such as frequently occur in connections of spandrel beams and wall girders to columns, this eccentricity should be considered in the calculations.

To facilitate the erection, connections of beams to columns should always be by a shelf having the proper shear angles under, rather than by side connections. Another advantage in this form of connection is that the deflection of the beam does not cause so much bending stress in the column. As will be seen from Fig. 106, if a deep beam or girder were connected by angles in the web, a deflection in the beam would cause the top to tend to pull away from the column; and, if the beam were held rigidly by side angles, considerable bending stress in the column would result.

Selection of Sections. The particular form of column section will vary with the conditions.

1. The first consideration is usually the amount of load; certain forms cannot be used without excess of metal if the loads are light; and conversely, certain other forms cannot be used economically if the loads are very heavy.

2. The next point to be considered is the way the beams come to the column. If the framing is symmetrical and on four sides, any of the sections could be used; in such a case, however, it would be simpler to avoid single or double angles for use as columns.

If the connections are eccentric, then a section stronger in the direction of eccentricity should be chosen, and one that will admit of easy connections. If a heavy girder comes in on top of a column, then the metal must be specially arranged to meet this condition. The consideration of these points will be taken up and illustrated in detail under the head of "Connections."

3. In the case of wall columns, the architectural details,—such as size of pier, relation to ashlar line, thickness of walls, etc.,—by limiting the dimensions of column, generally affect the choice of form of section.

4. Other architectural conditions, such as, shape and size of finished column, relations to partitions, provision for passage of pipes, wires, etc., have to be considered in the general choice, as it is desirable to adopt the same type throughout even if the limitations affect only certain columns.

5. The condition of the steel market as regards delivery of certain shapes within the required time, is always a factor. A delay of several months may sometimes be saved by proper consideration of this point.

Calculation of Sections. The type of column having been decided on, the calculation of sections is the next step.

The effect of connections is as important in the case of cast-iron columns, as in that of steel columns, and typical details are shown in Plates X and XI, Figs. 108 to 111.

Plate XII, Fig. 112, shows a cast-iron ribbed base designed for a square column similar to that shown by Fig. 110.

Fig. 113 shows a cast-iron base designed for a steel column, the section of which is indicated by the hatched lines. An important feature of all cases of this type is to have the metal arranged so as to conform to the metal of the column that rests upon it.

A good many designers give a slight pitch downward to the brackets forming the seats of beams. This is of advantage in avoiding the tendency, which would otherwise occur, of the beam to bear most heavily on the other edge when deflection under loading takes place.

There are several types of column formulæ in general use; and, as noted under "Building Laws and Specifications," there is a variation in the legal requirements of different cities in this respect.

Gordon's formula is perhaps the oldest and most generally used. This is as follows:

$$f = \frac{12500}{1 + \frac{l^2}{ar^2}}$$

where f = safe fibre strain reduced for length and radius of gyration;

l = unsupported length, in inches;

r = radius of gyration, in inches;

a = a constant, of the values below:

= 36,000 for square bearing;

= 24,000 for pin and square bearing;

= 18,000 for pin bearing.

Plate X. Cast Iron Columns

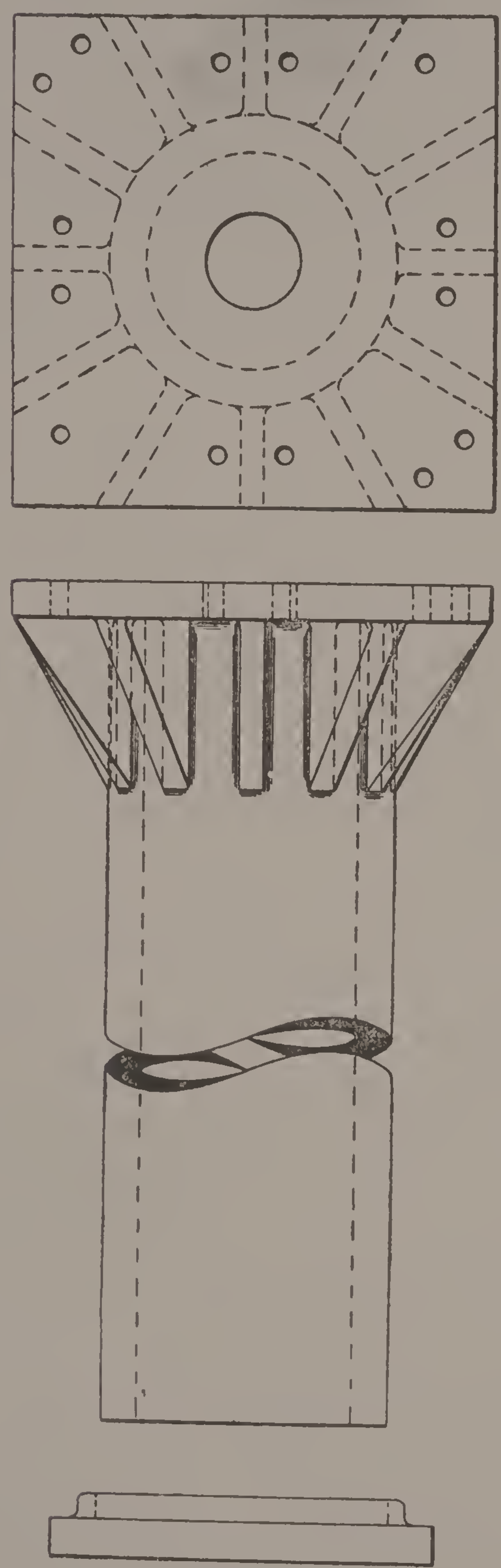


Fig. 108

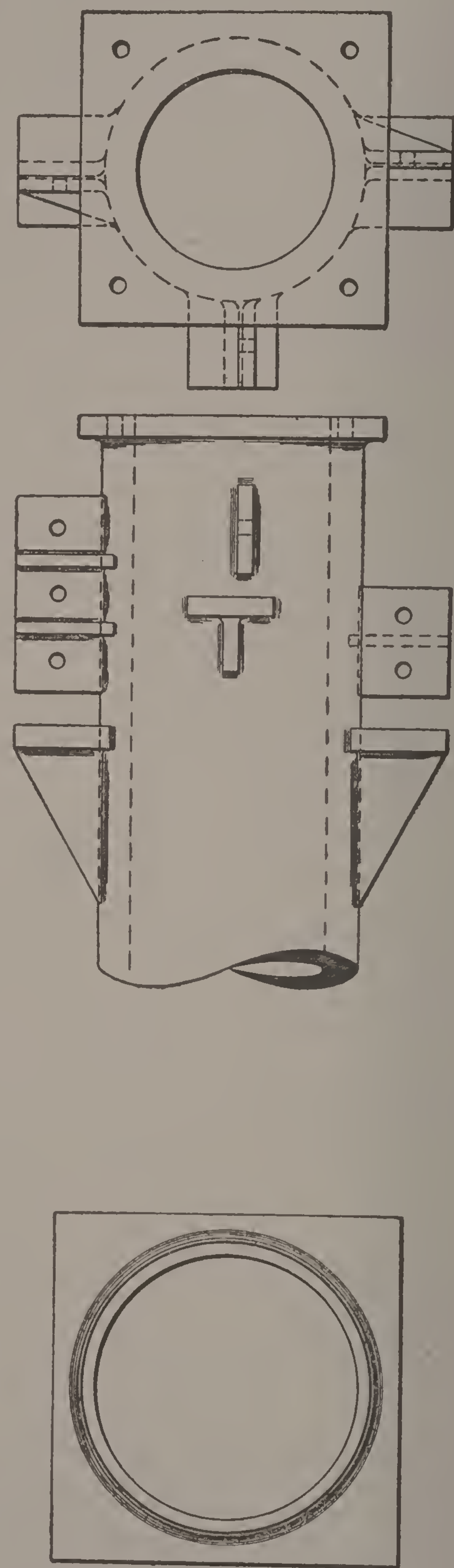


Fig. 109.

Plate XI

Cast Iron Columns

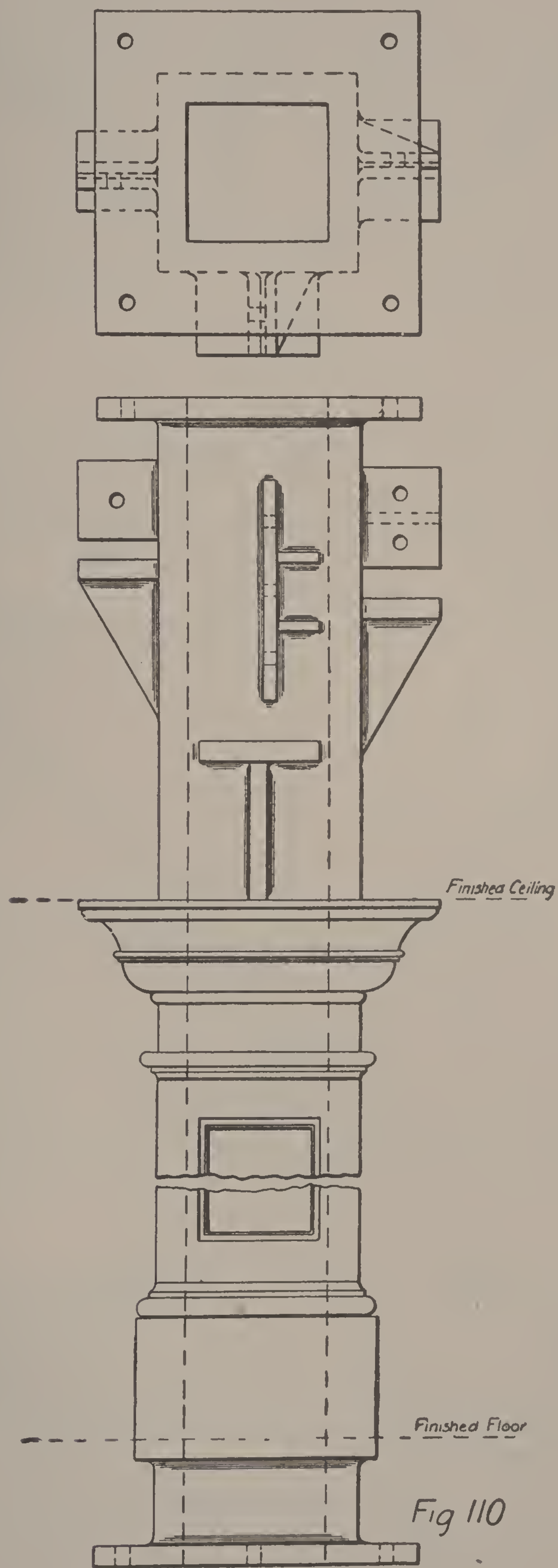


Fig 110

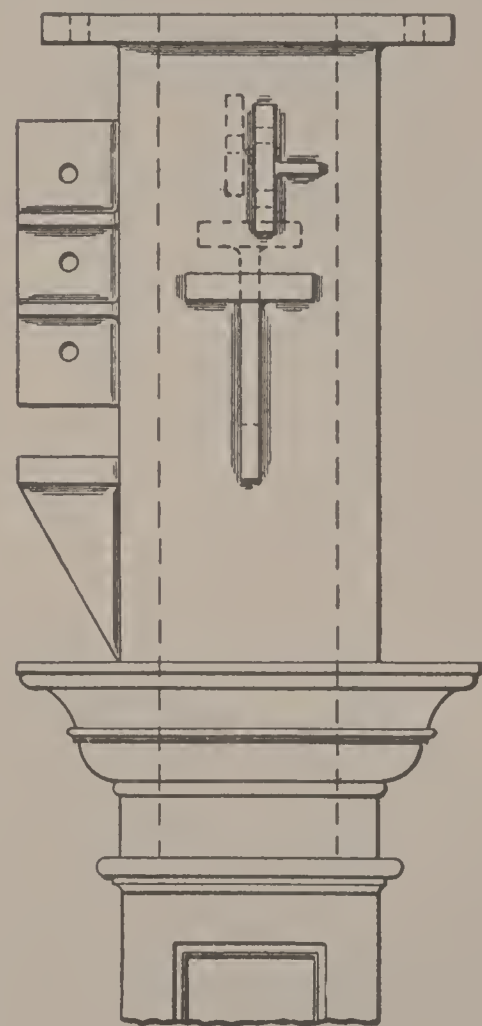


Fig. 111

The formula used by the Carnegie Steel Company for the calculation of capacity of Z-bar and box-section columns is as follows:

$f = 12{,}000$ for lengths of 90 times the radius of gyration.

$f = 17{,}100 - 57\,\dfrac{l}{r}$ for lengths greater than above.

Cooper's formula is as follows:

$$f = 16{,}000 - 58\,\frac{l}{r}.$$

This formula, while similar in form to the one used by the Carnegie Company for lengths above 90 radii, is applied by Cooper to all lengths.

The American Bridge Company use the following formula for all lengths:

$$f = \frac{17{,}000}{1 + \dfrac{l^2}{11{,}000\,r^2}}.$$

The results given by these formulæ vary considerably, the variation increasing under certain conditions of length and of radius of gyration, and being greater with large values in ratio of length to radius of gyration.

The student should work out the areas of column required by these formulæ for different values of $\dfrac{l}{r}$, to become familiar with their differences.

Columns, Diagrams, and Tables. The most useful diagram for the calculation of capacity of columns and of required areas under concentric loading is one which gives the allowable unit-stress according to the formula to be used. Such a diagram would be made by laying off vertical ordinates representing different values of radius of gyration, and horizontal ordinates representing length of column in feet. On this diagram curves could be plotted, corresponding to a number of formulæ.

Plate XII

Cast Iron Ribbed Bases.

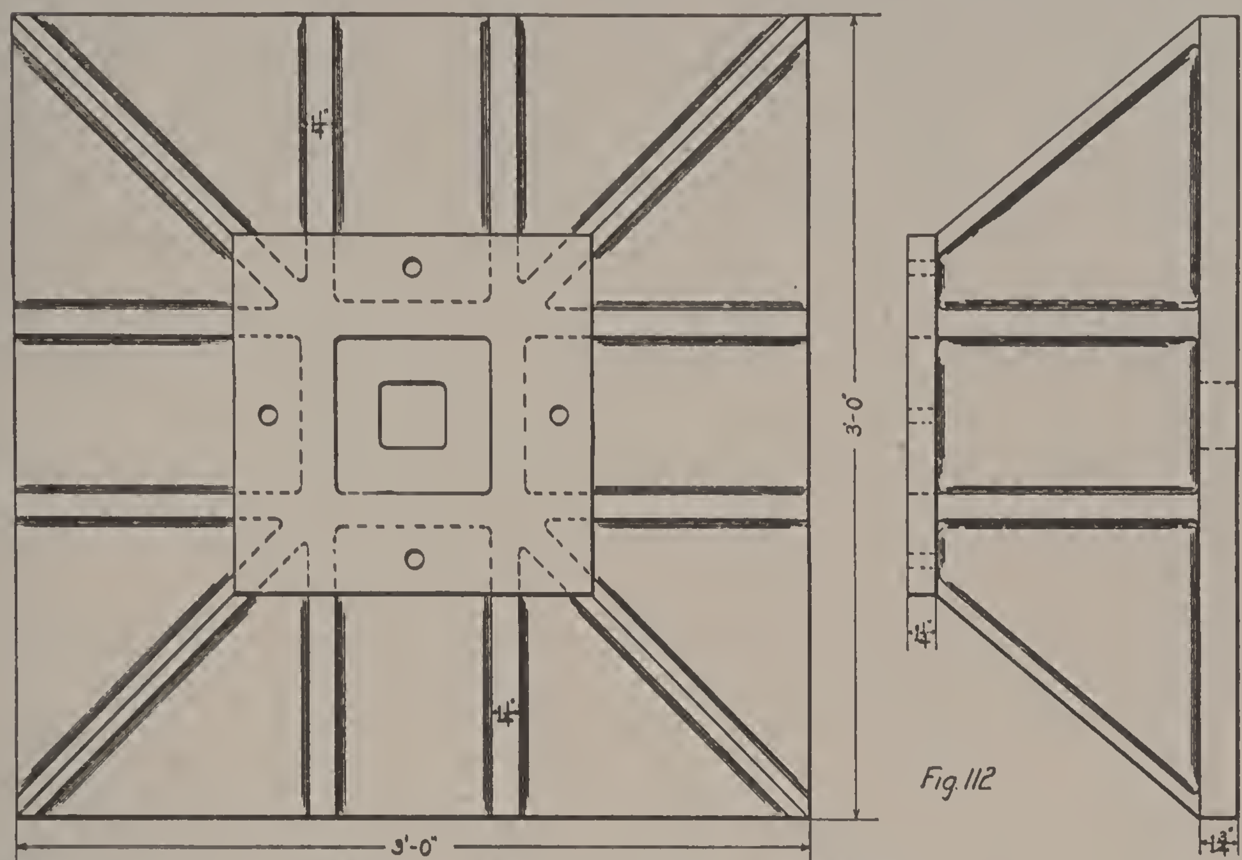


Fig. 112

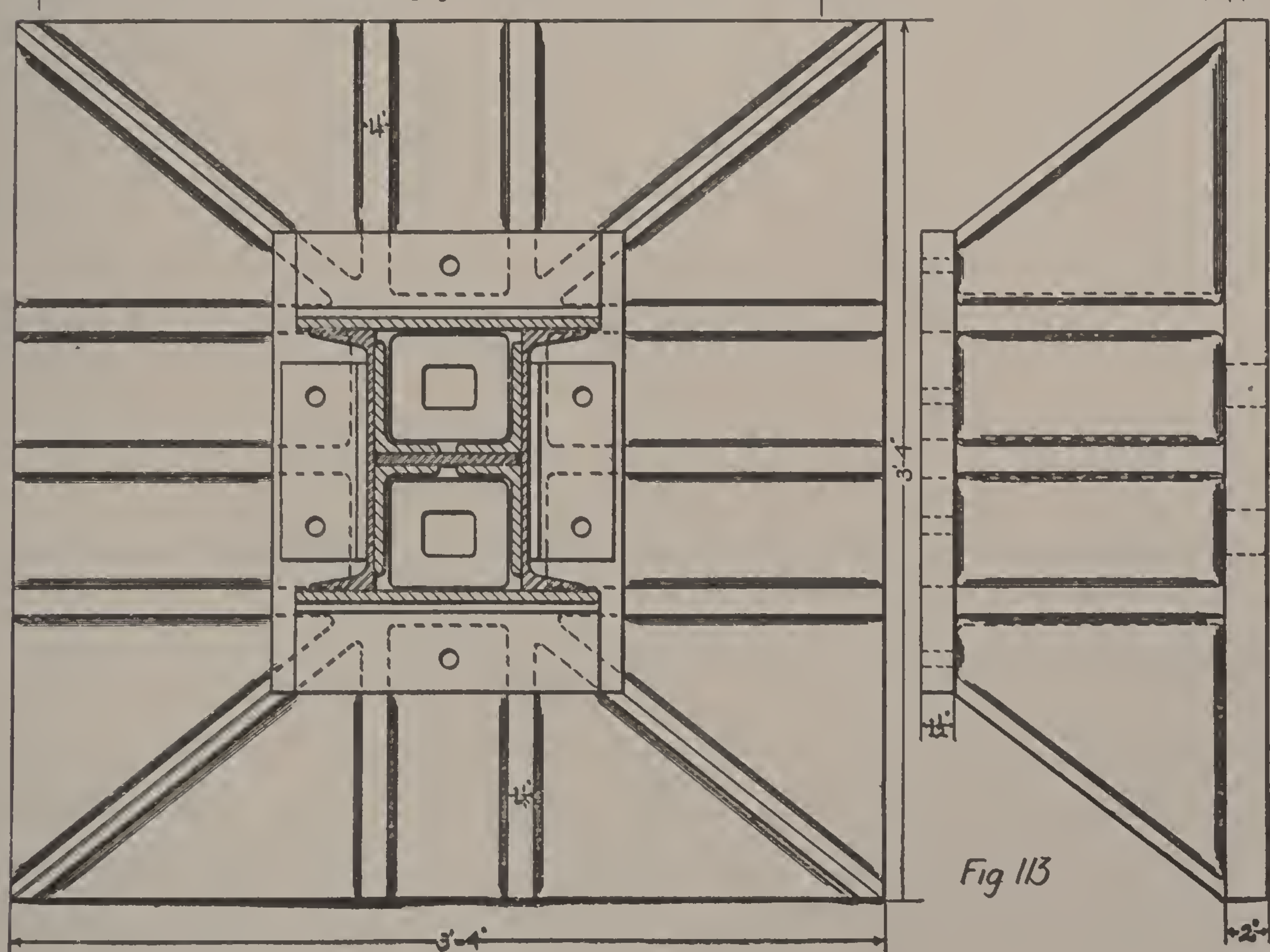


Fig 113

In practice this diagram would be used as follows: Assume a certain section which the judgment of the designer indicates as approximately correct. Calculate the radii of gyration, and, this

Plate IX. COLUMN SCHEDULE

STORY HEIGHTS		COLS. NOS 1, 2, 3, 4, 7, 9, 11, 12, 13, 17, 18, 20, 21, 22, 26	COLS. NOS 5, 6, 8, 10, 14, 15, 16, 19, 23, 24, 25, 27, 28, 34	COLS. NOS 29, 30, 31, 32, 33, 35, 36
	Roof			
Variable	11th	Web 10"×5/16" 4Ls. 3½"×3"×5/16" Area=10.85□" Load=36 Tons	Web 8"×5/16" 4Ls. 3½"×3½"×½" Area=16.00□" Load=26 Tons	Web 10"×5/16" 4Ls. 3½"×3"×5/16" Area=10.85□" Load=51 Tons
10'-7"	10th			
10'-7"	9th	Web 10"×½" 4Ls 4"×3"×3/8" Area=14.92□" Load=79 Tons	Web 10"×3/8" 4Ls 3"×3"×3/8" 2pls. 8"×½" Area=20.19□" Load=63 Tons	Web 10"×½" 4Ls 4"×4"×½" Area=20.00□" Load=108 Tons
10'-7"	8th			
10'-7"	7th	Web 10"×½" 4Ls 4"×3"×3/8" 2pls. 10"×3/8" Area=22.42□" Load=119 Tons	Web 10"×½" 4Ls 4"×3"×3/8" 2pls. 10"×7/16" Area=23.67□" Load=100 Tons	Web 10"×½" 4Ls 4"×4"×½" 2pls 10"×7/16" Area=28.75□" Load=163 Tons
10'-7"	6th			
10'-7"	5th	Web 10"×½" 4Ls. 4"×4"×½" 2pls 10"×3/8" Area=27.50□" Load=160 Tons	Web 10"×½" 4Ls 4"×4"×½" 2pls 10"×½" Area=30.00□" Load=136 Tons	Web 10"×5/8" 4Ls 4"×4"×5/8" 2pls 11"×3/8" Area=38.44□" Load=218 Tons
10'-7"	4th			
10'-7"	3rd	Web 10"×5/8" 4Ls. 4"×4"×5/8" 2pls. 10"×½" Area=34.69□" Load=199 Tons	Web 10"×5/8" 4Ls. 4"×4"×5/8" 2pls. 11"×9/16" Area=37.07□" Load=173 Tons	Web 10"×¾" 4Ls. 5"×5"×5/8" 2pls. 11"×¾" Area 47.44□" Load=272 Tons
12'-1"	2nd			
13'-8"	1st	Web 12"×¾" 4Ls. 4"×4"×5/8" 2pls 10"×5/8" Area=39.94□" Load=239 Tons	Web 12"×¾" 4Ls. 4"×4"×5/8" 2pls. 11"×¾" Area=43.94□" Load=210 Tons	Web 12"×¾" 4Ls 5"×5"×7/8" 2pls. 11"×¾" Area=57.46□" Load=327 Tons
11'-3"	Basement			

Fig 107

having been done, the allowable fibre strain, for the least ratio of length and radius of gyration can be taken from the diagram.

If the area as determined by this allowable fibre strain varies materially from that of the assumed section, a new assumption must be made and the process repeated.

Problem. Plot on cross section paper which is divided into spaces $\frac{1}{10}$ inch square, a column diagram as described above, and draw curves for each of the formulæ given ; ordinates to provide for radius of gyration from 0 inches to 8 inches, and of length in feet from 0 feet to 60 feet. The scale to be $\frac{2}{10}$ in. = 1 ft., and $\frac{2}{10}$ in. = 1 in. radius.

Tables or diagrams are also made of the safe capacity of different column sections for varying lengths, as, for instance, those given for Z-bar columns and for channel and plate columns. Similar data could be prepared for other types of column ; but unless the designer were working under one column formula constantly, such tables, in order to be useful, would need to be made applicable to all formulæ, and would, therefore, involve considerable time in their preparation.

The column loads should be tabulated with the sections of columns as illustrated by Plate IX, Fig. 107. These loads are the reactions from the different beams framing into them.

Practical Considerations. In general it is the practice to vary the section of column only at every other floor. The reason for this is that the saving in number of pieces to handle and to erect, and in splices, and the gain in time of delivery, more than offset the extra metal added in one story.

In some cases, also, it is advisable to adopt a uniform dimension column so as to avoid changes in length of beam from story to story that would be necessitated by even slight changes in size of column. In special cases many other practical points are likely to arise, which, by affecting rapidity of preparation of drawings, or of shop work, or of erection, may make it advisable to adopt certain forms, or may affect the theoretically economical section. The successful designer is the one who can foresee all these considerations and properly weigh their effect.

Cast-Iron Columns. Where the conditions are such as to require a rigid frame, and consequent stiffness in joints and connections, it is not advisable to use cast-iron columns, because connections to such columns must always be by means of bolts,

which are apt to work loose and which never fit the holes perfectly. Furthermore, cast-iron columns are ill adapted to resist lateral deflection. Their use, therefore, should be confined to buildings of moderate height and in which the walls themselves furnish all necessary stiffness.

In order to use the formulæ for strength of cast-iron columns, given in Table 10 of Part I, the ends must be turned true. If this is not done not more than one-half their values should be used.

Concrete and Steel Columns. Considerable attention has been given of late to the strength of steel and concrete columns. Some systems have already been proposed, in which columns composed of rods imbedded in concrete are used. Such construction has been used to some extent for chimneys, and in a few buildings. It is also suggested that in certain classes of buildings, notably mills and manufactories, the steel members now quite commonly employed for columns could be encased in a solid and substantial envelope of concrete, and that this casing not only would have the advantage of fireproof protection, but, by the added stiffness afforded the columns, would enable higher fibre strains to be used in the design of the steel members, and would thus result in better and cheaper construction.

PROBLEMS.

1. Determine by use of the column diagram described in the problem above, the proper section of plate and four-angle column to carry a girder over the top, bringing to the column a load of 100 tons. Unsupported length of column 18 feet. Use Gordon's formula.

2. In the above problem substitute for a plate and angle column a box column composed of channels and side plates, and determine proper section by use of Carnegie formula and American Bridge Company formula.

3. Given a column built into a 16-inch brick pier and loaded with 125 tons. Required the proper section of plate and angle column, assuming column to be stiffened by wall in direction of wall. Length 16 feet. Use Gordon's formula.

4. Given a column loaded as shown by Fig. 114. Determine proper section of plate and angle column, using Gordon's formula.

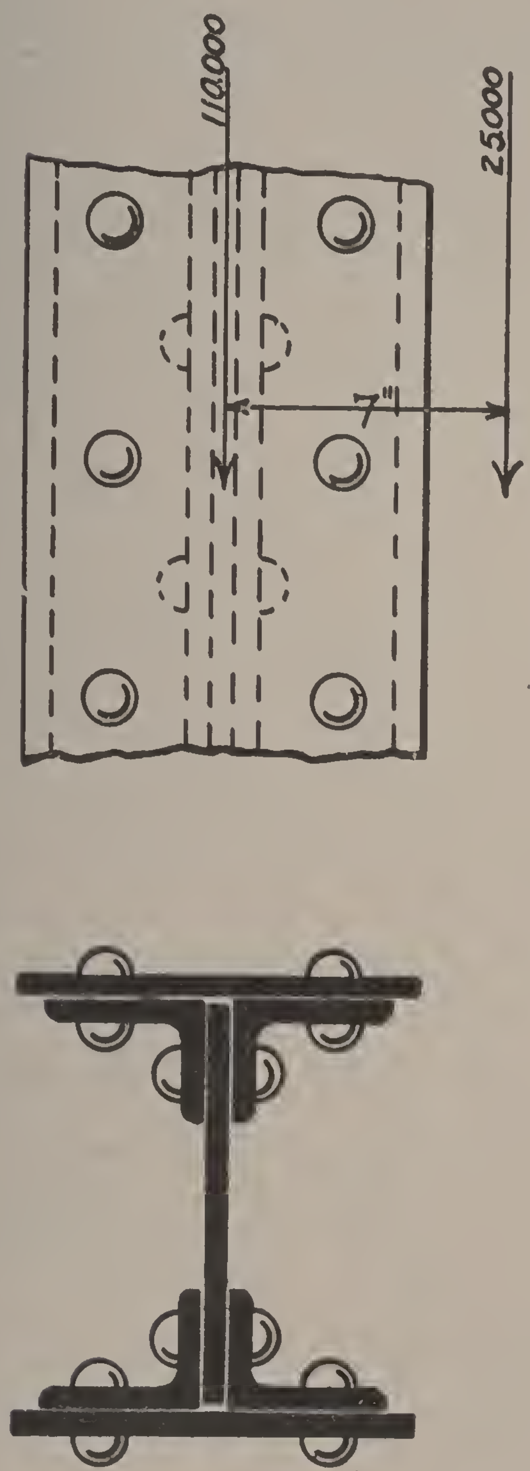


Fig. 114

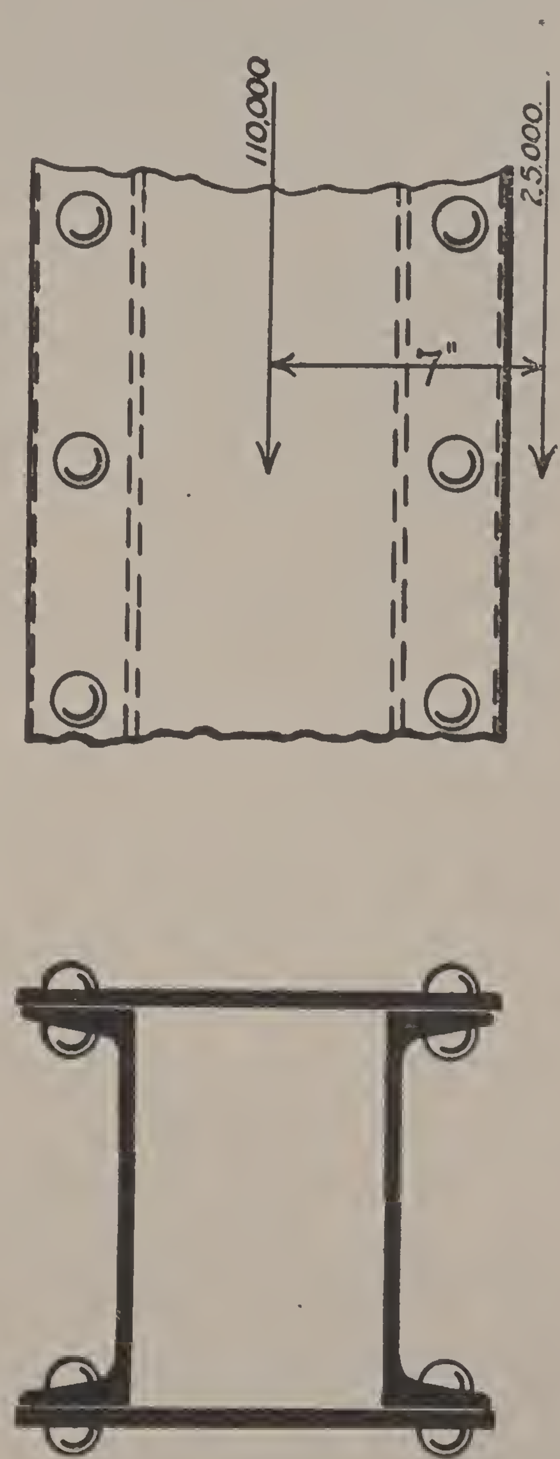


Fig. 115.

5. Given the same column as above, but with the axis of column at right angles to previous position, as shown by Fig. 115. Determine required section of column using channels either latticed or with side plates. Use Gordon's formula.

TRUSSES.

For spans under 35 feet, a riveted or beam girder is ordinarily more economical than a truss, unless the conditions of loading are peculiar.

Selection of Type. The type of truss selected depends generally upon (1) span, (2) pitch of roof, (3) covering of roof, (4) available depth, (5) load to be carried.

All the above considerations affect jointly the choice of type; no single type would be used under certain lengths of span, for instance, with different combinations of the other conditions. A short span and flat roof might lead to a lattice truss, but if the roof had a steep pitch another type would be used.

The covering of the roof affects the position and number of panel points, and therefore the type. If the planks rest directly on the top chord of trusses, then the panels can be arranged as may be most economical. If the roof is of corrugated iron, the size of sheets will limit the spacing of purlins, and, as these should come at the panel points, this will determine the number of panels.

The position of a monitor or skylight would also largely determine the number of panels.

If the depth is limited, then certain types cannot economically be used. If there is a ceiling or shafting to be carried, or any other conditions making a horizontal bottom chord essential, then this must be provided.

In almost all cases, therefore, there are certain conditions that determine arbitrarily certain features of the truss, and these indirectly fix the type that should be used.

On pages 109 and 110 are given types in general use, and a consideration of the points noted above will illustrate their application to these types.

Bracing. An important feature in all trussed roofs is the bracing. Trusses cannot be economically designed without supporting at intervals the top chord against lateral deflection. As was noted in the case of beams, the allowable fibre stress must be reduced with the ratio of length to radius of gyration.

This support is given by the plank if directly attached to the truss, or by purlins. Such purlins should be efficiently connected to the truss. If the conditions of framing are such that the regu-

lar construction does not hold the truss, then special steel bracing must be used. In the case of very large roofs, special steel bracing should always be used, as there would not be sufficient stiffness in the connections of purlins to properly brace the trusses.

Such bracing is generally of the kind known as X bracing, alternate panels of adjacent trusses being connected by angles or rods. Not every bay is braced, but every other bay, or a less number, depending on conditions.

Considerations Affecting Design of Trusses. Light trusses are subject to distortion in shipping, handling and erection. To guard against such distortion it is sometimes important, therefore, to provide more than the strength calculated for vertical loads when the truss is in position.

In designing a roof, certain features that affect the weight of a truss can often readily be avoided. Some of these are indicated as follows:

Long web members should be arranged so that the stress will be tension, not compression.

It is not economical to use a double system of web members, such as a lattice truss, except in the case of light loads and shallow depth.

No web members should be provided that do not take direct load or are not needed for support of the chords.

Concentrated loads, such as purlins, or hangers, etc., should, if possible, come at panel points, as otherwise the bending stress in the chords increases materially the weight of truss.

The roof plank resting directly on the top chord of truss increases the weight of truss, but the saving in purlins sometimes offsets this.

The spacing of trusses should, if possible, be such as will develop the full strength of the members of the truss. In some cases the conditions are such that the lightest sections which it is practicable to use are not strained nearly to their capacity.

Practical Considerations. Trusses are generally riveted up complete in shop and shipped whole, unless it is impracticable to do so. Not only is riveting in the field expensive, but the rivets are not so strong, being generally hand-driven instead of power-driven.

In some cases it is not practicable to rivet the trusses complete, on account of their size. If they are to be shipped by railroad, it is always necessary to be sure that they do not exceed the limits of clearance necessary along the route they have to traverse.

These limits have to be obtained in each special case, as the clearances of bridges and heights of cars vary. This consideration sometimes makes it necessary to ship all the parts separately and to rivet in the field, or to make one or more splices of the truss as a whole. The weight of trusses, with regard to the rigging available for handling and transporting them, has also to be considered.

During the process of erection it should be remembered that in the design of the truss the lateral bracing of the completed structure is generally figured on, and until the structure is complete, ample temporary bracing should be provided. Many failures of roofs are due to neglect of this precaution.

Determination of Loads. The loads for which a roof truss should be figured are: the dead weight of all materials; an assumed snow load, varying with the latitude and slope of roof; a wind load, varying with the slope of roof; a ceiling load, if there is to be any; and such other special loads as may occur in particular cases.

Snow varies from 12 to 50 pounds per square foot of roof, according to the degree of moisture or ice in it. On a flat roof an average allowance for snow is 30 lbs. per square foot of roof. A roof sloping at an angle of 60° to the horizontal would not generally need to be figured for snow, unless there were snow guards to keep the snow from sliding off.

The wind is assumed to blow horizontally, and the resulting horizontal pressure is generally taken at 40 lbs. per square foot. The normal pressure with different slopes on this basis is indicated in the following table:

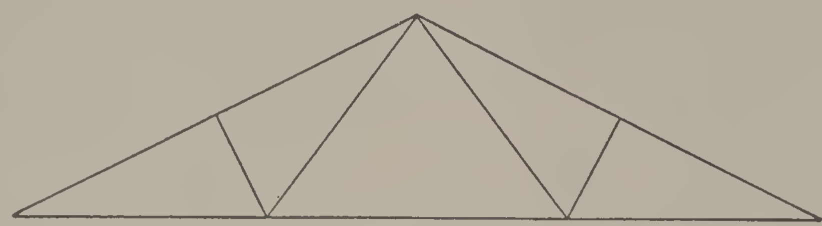
TABLE XVIII.

Roof Pressures.

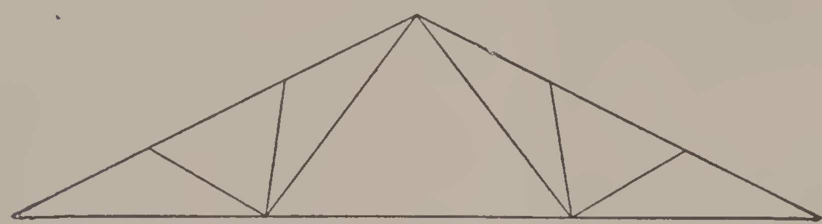
In pounds per square foot, for an assumed horizontal wind pressure of 40 lbs. per square foot.

Angle of Roof with Horizontal	5°	10°	20°	30°	40°	50°	60°	70°	80°	90°
Pressure Normal to Surface of Roof	5.0	9.6	18.0	26.4	33.2	38.0	40.0	40.8	40.4	40.0
Pressure on Horizontal Plane	4.9	9.6	16.8	22.8	25.6	24.4	20.0	14.0	6.80	0
Pressure on Vertical Plane	0.4	1.6	6.0	13.2	21.2	29.2	34.0	38.4	39.6	40.0

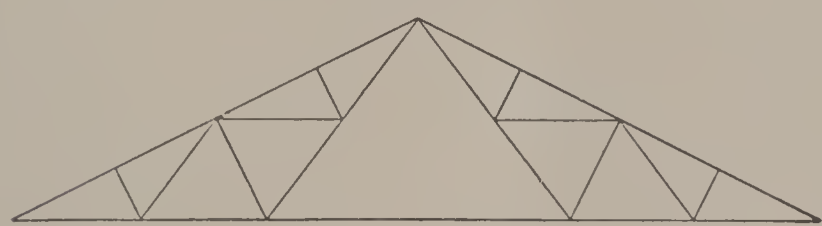
In the calculation of the maximum strain, the combinations of dead load, snow load, and live load should be considered. It is not necessary, however, to consider the wind and snow acting on the same side at the same time as a wind giving the assumed pressure would blow all the snow off this side. Wind on one side and



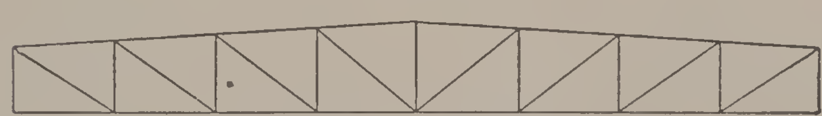
2 PANEL TRUSS.
Fig. 116.



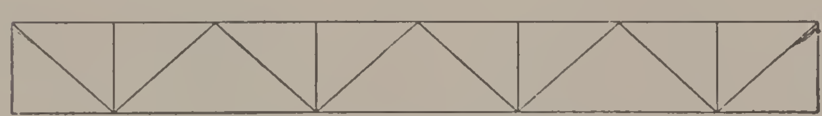
3 PANEL TRUSS.
Fig. 117.



4 PANEL TRUSS.
Fig. 118.



FLAT PITCH ROOF TRUSS.
Fig. 119.



PARALLEL CHORD ROOF TRUSS.
Fig. 120.

snow on the other side, or snow on both sides, generally give the maximum live-load strains.

The total dead and live loads should not be taken as less than 60 lbs. per square foot, and, in general, the conditions render allowance for a greater total load necessary.

The design of trusses will be taken up in the course on Theory and Design.

CONNECTIONS AND DETAILS OF FRAMING.

Figs. 121 to 126 show types of connections of beams to girders and columns. Connections to girders are nearly always of these standard forms, which are the Carnegie standards. In certain cases, individual shops have forms that vary slightly from these, but not to any great degree. It is essential to use the standard form wherever possible because these connection angles are always kept in stock, and the shop work of laying out and punching the material is thereby much simplified. Conditions of framing sometimes arise requiring special connections, but these should always be avoided if possible. In the smaller shops, an extra charge is generally made for coping beams so that where practicable, without increasing the cost of other portions of the work, it is better to frame beams far enough below girders to avoid this coping. The larger shops, however, are so equipped that this coping does not involve an extra operation, and a beam that must be cut to exact length, and has framing angles, can be coped without extra charge.

Connections of beams to columns where they frame centrally with the columns are of the general type shown by Figs. 95 to 105. The exact size of angles varies somewhat with the column section, because the riveting in the framing angles must conform to the spacing required for punching the members of the column. If the beams frame into the column eccentrically, no standard forms can be followed, but each case must be treated individually. Plate and box girders framing into other girders are generally connected by angles riveted to the webs, because ordinarily the depths of the girders will not allow shelf angles underneath. Where such girders frame to columns, however, it is better to use shelf angles with stiffener angles, or shear angles as they are generally called, because this facilitates the erection by providing a seat upon which the girders can rest when swung into position, and also because side connections would cause bending stresses in the column, as noted on page 112.

Column Caps, Bases and Splices. Where heavy girders or a number of beams come over the top of a column, the column section should be made up of such shapes and of such size that the metal of the column comes as nearly as possible under the metal

STANDARD BEAM CONNECTIONS

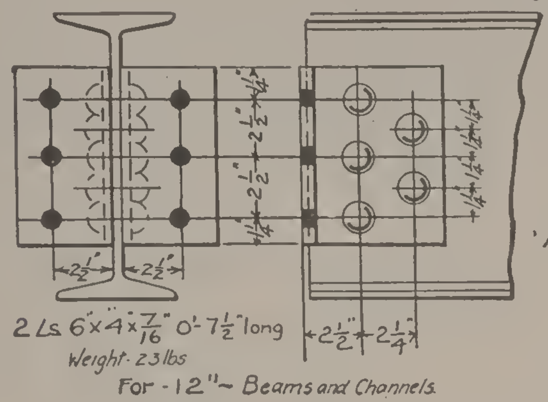


Fig. 121.

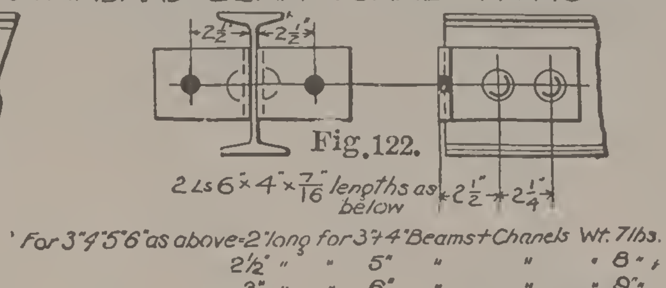


Fig. 122.

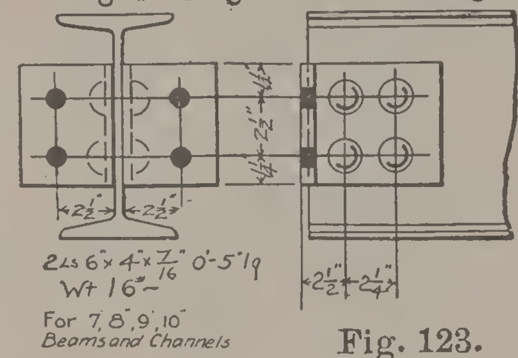


Fig. 123.

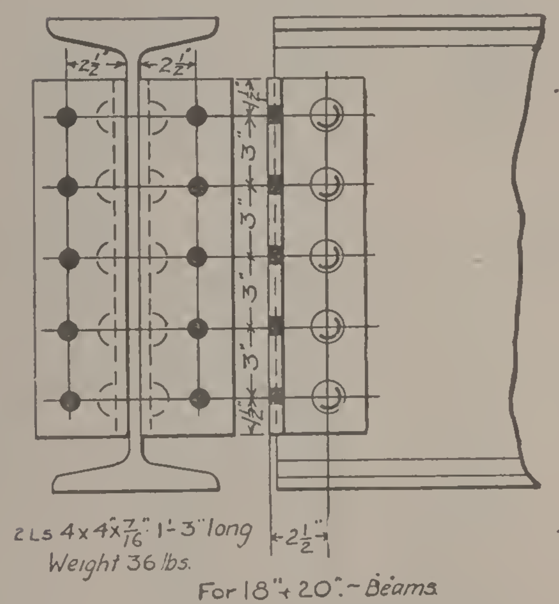


Fig. 124.

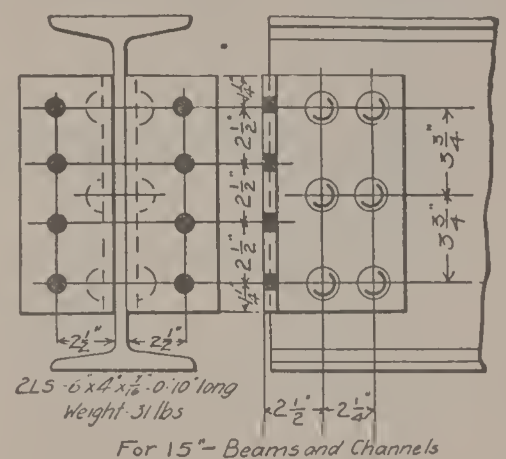


Fig. 125.

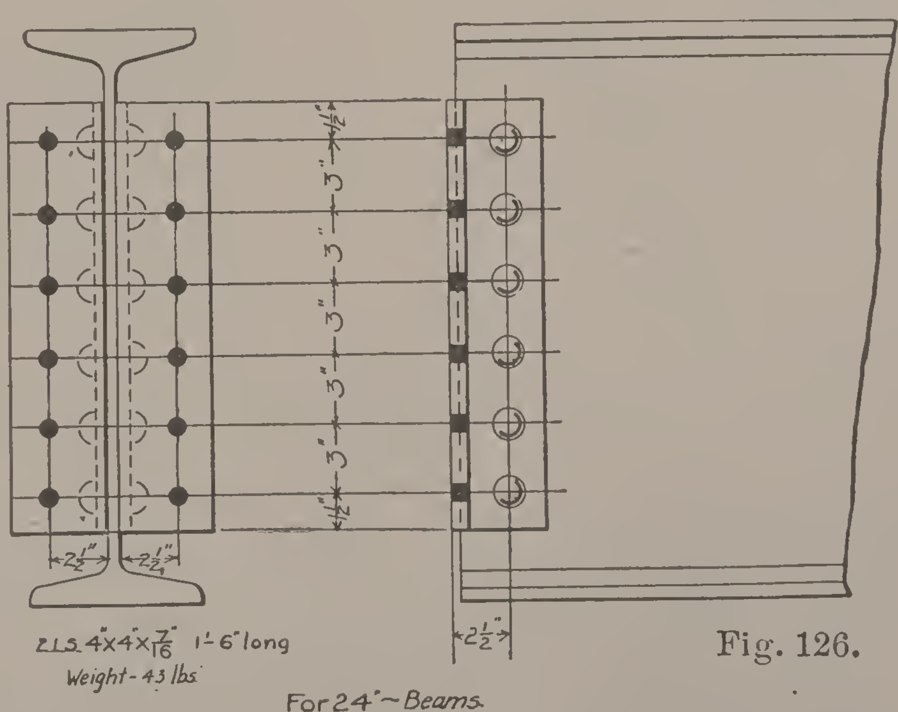
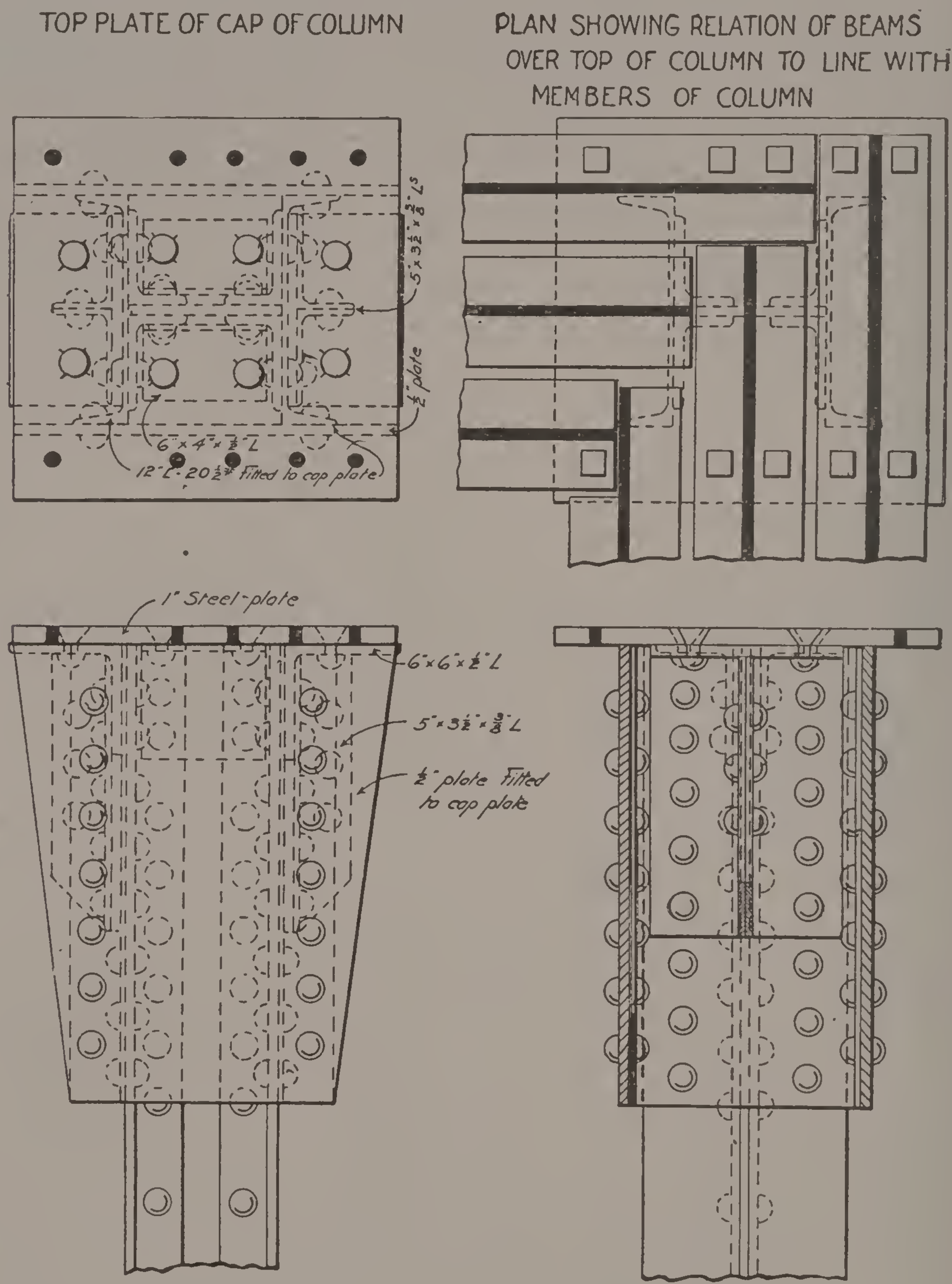


Fig. 126.

of the beams or girders. If the girder has stiffener angles over the bearing, as it generally does, shear angles should be put in the column directly underneath. The webs and stiffener angles of

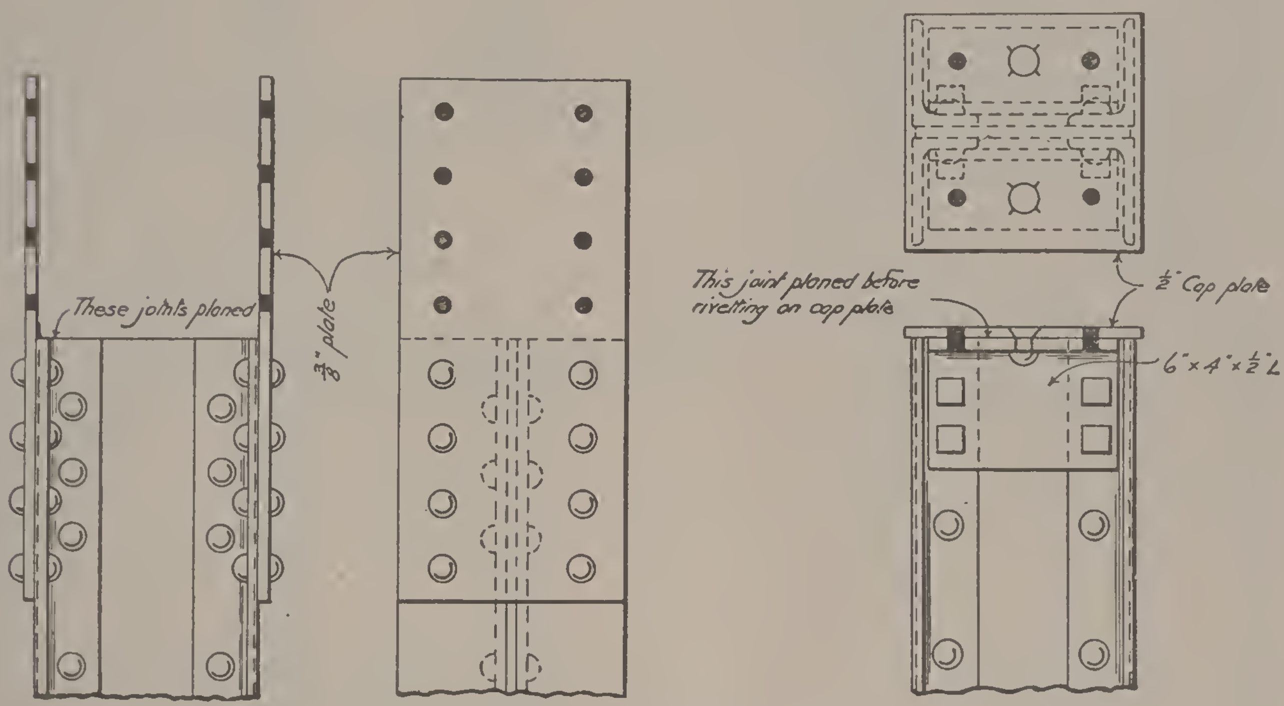


SPECIAL CONNECTION-3 BEAM GIRDER OVER TOP OF COLUMN.
Fig. 127.

the girders or beams should not bear on an unsupported cap plate, but this cap plate should be well supported by shear plates or angles. The above is illustrated by Fig. 127.

Column splices are not ordinarily designed to carry the full load of the upper section through the splice to the lower section. Such design would result in splices of considerable length, which

in some cases would be difficult to arrange and always expensive. The general practice is to have the top of the section below and the bottom of the section above the joint planed to a true surface so that there will be a perfect bearing between them. If this is done, the load is transmitted from section to section by direct compression just as in the body of the column. However, the splice



TYPES OF COLUMN JOINTS.

Fig. 128. Fig. 129.

should be designed to give the column the full strength of the uncut section as regards stiffness against lateral deflection. As the splice is near the floor beam connections, where the column is braced laterally, this can generally be easily accomplished. Types of column splices are shown by Figs. 128 and 129.

Fig. 130 illustrates a connection to column of a beam located eccentrically with regard to the column.

Such connections require an extra number of rivets in addition to those required for the direct load in order to resist the tendency to rotation due to the eccentricity.

Some of the special types of framing which occur are shown by Figs. 131 to 140.

Where a beam comes below another beam, as shown in Figs. 131 and 132, a connection such as shown can be used. If the load coming on the hanger is such as to require something stronger than a channel, a simpler connection will result by using two

channels spread far enough for the connection plate to be riveted between, as shown by Fig. 132, instead of a beam.

A three-beam girder framed to another beam is shown in Fig. 133. The inside beam can have angles on each side of the web. This beam must be placed before the outside beams in order to

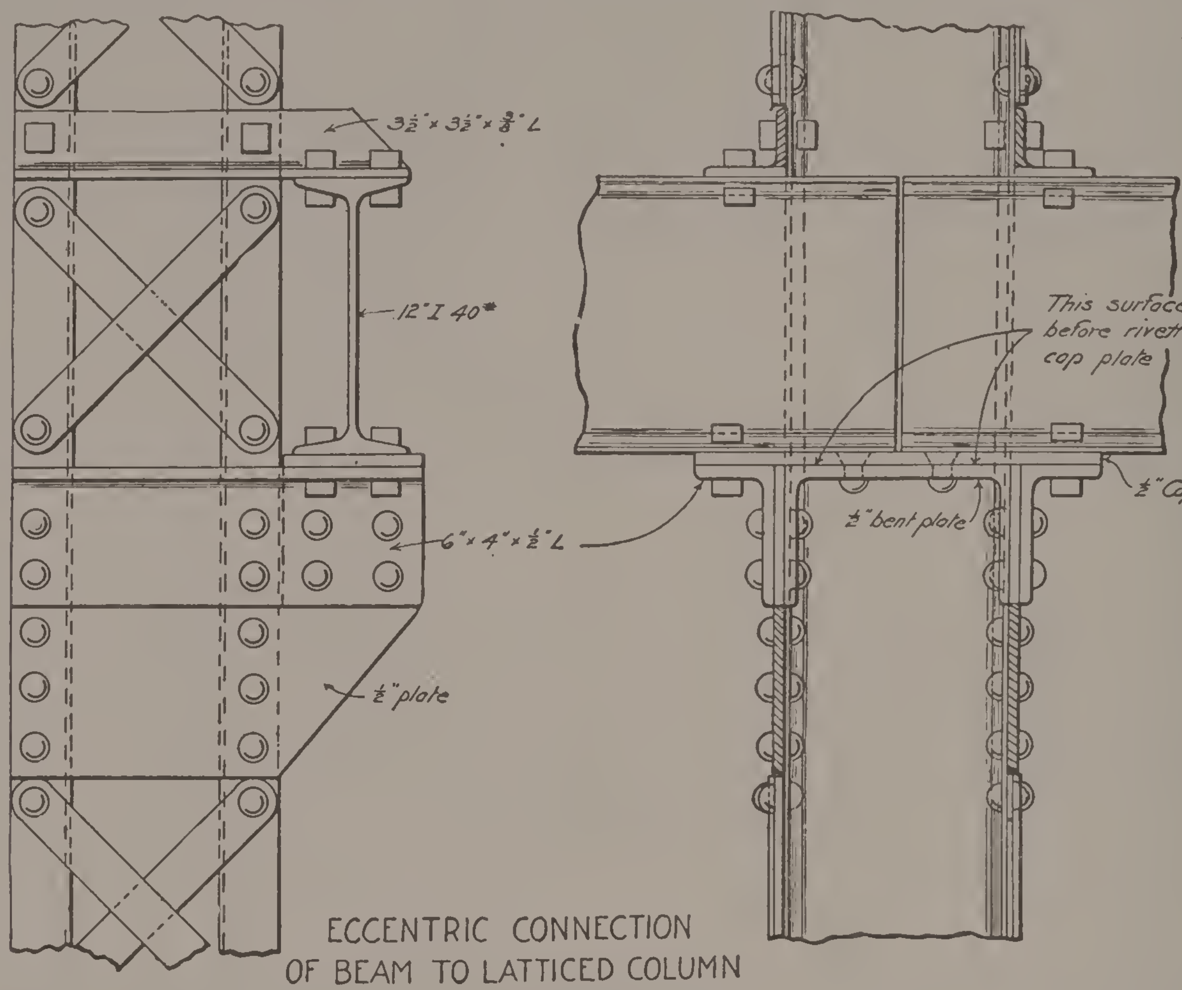


Fig. 130.

make this connection. Unless the three beams are spread a considerable distance apart, the outside beams can have an angle on only one side of the web; this angle therefore should be a 6″ × 6″ angle in order to get the same number of field rivets as with two standard angles.

Fig. 134 shows a beam dropped below the the top of a 24-inch beam girder to which it is framed. With these large size girders it is often impossible to make a connection so that the beams will frame flush with the girder.

Figs. 135 and 139 show changes in the position of standard framing angles on the sides of webs of beams of different sizes framing on opposite sides of the same girder. These changes are necessary in order to use the same holds for both connections and to keep the connections standard.

DETAILS OF SPECIAL FORMS OF BEAM CONNECTIONS.

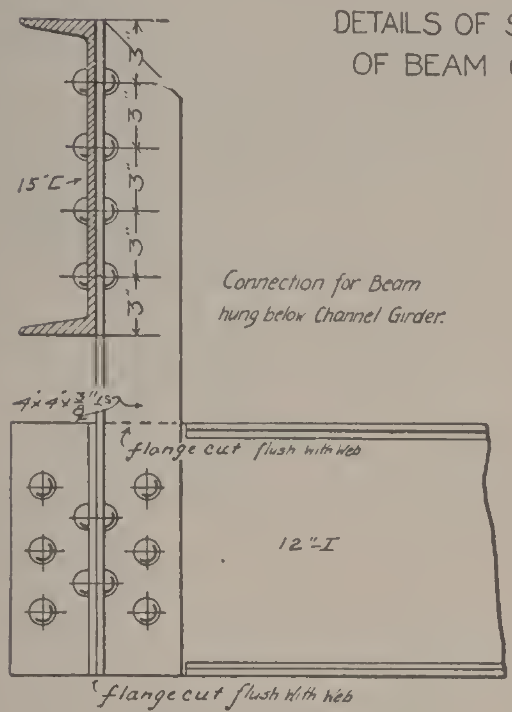


Fig. 131.

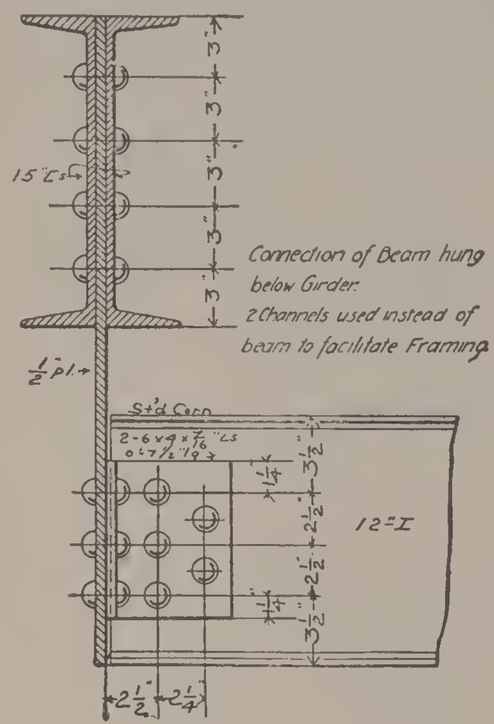


Fig. 132.

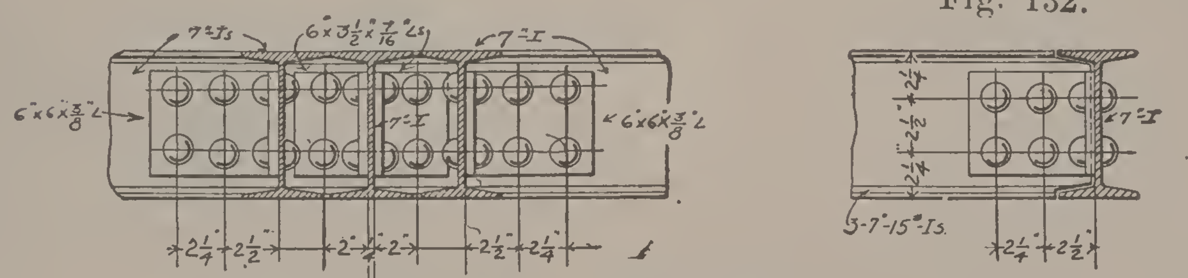
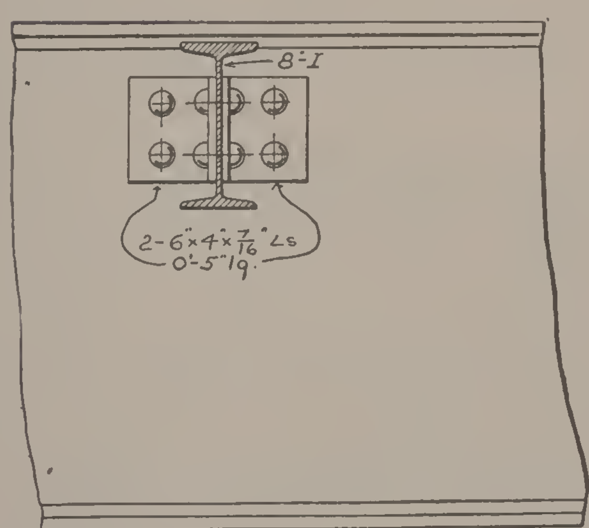


Fig. 133.



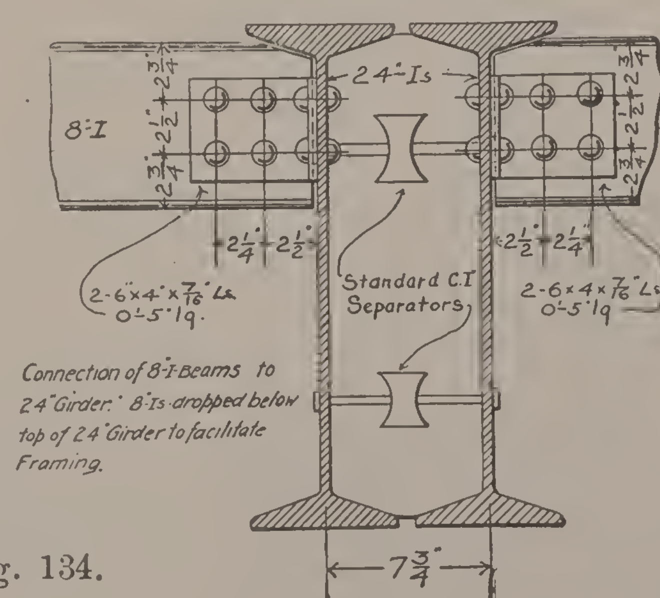


Fig. 134.

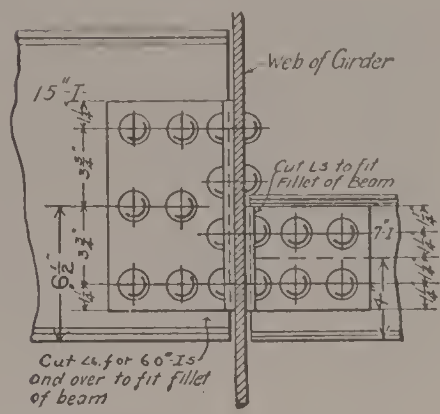
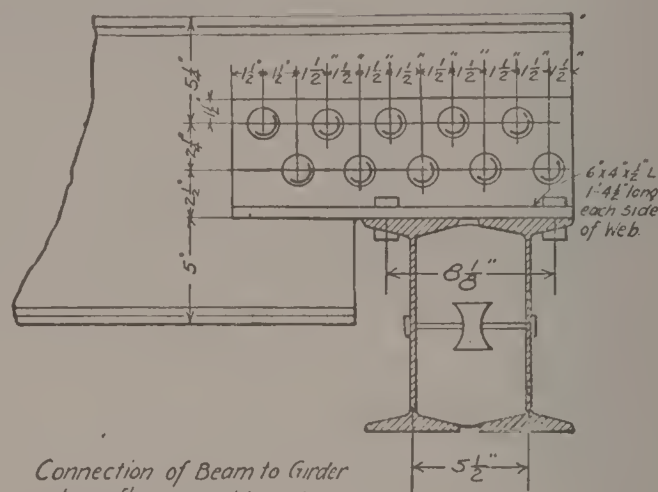


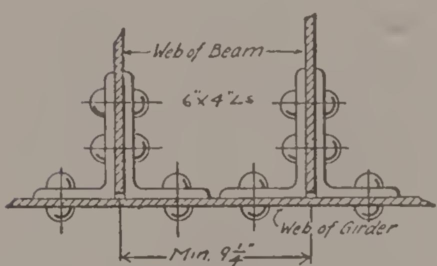
Fig. 135.

Connections for 15 and 7" Is framing opposite to each other.



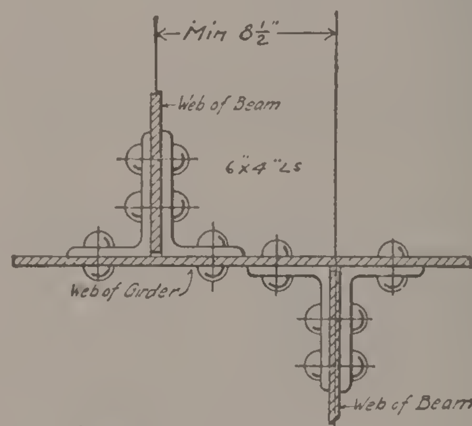
Connection of Beam to Girder where flange must be cut.

Fig. 136.



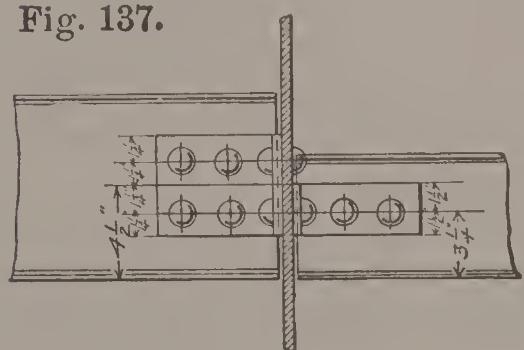
Beams framing on same side of Girder

Fig. 137.



Beams framing on opposite sides of Girder.

Fig. 138.



Connections for 6" Beam framing opposite to 10" Beam, 9" Beam or 8" Beam

Fig. 139.

TYPICAL SPECIAL CONNECTIONS.

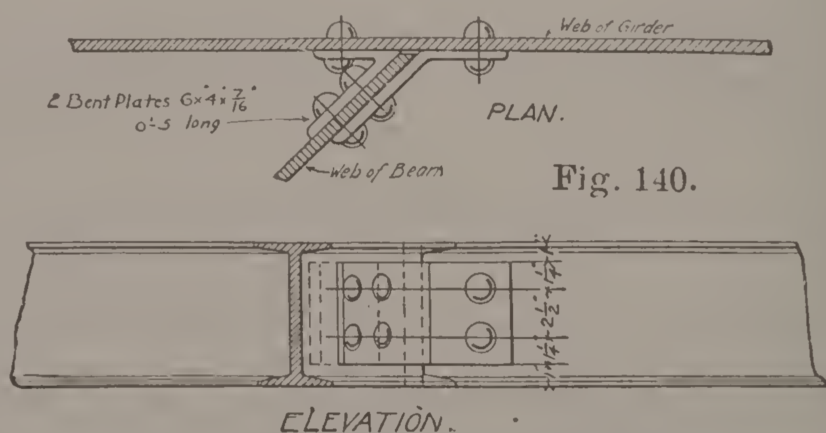


Fig. 140.

Connection for beam Framing to Girder on skew

Fig. 136 shows the connection of a beam framing partly below and above another beam where the lower flange has to be cut. In such cases the angles which are riveted to the web for a bearing should extend back on the web beyond the cut for a distance sufficient to get as many rivets in as are required for carrying the end shear. Therefore in this connection there should be at least twice the number of rivets required to carry the end reaction.

Figs. 137 and 138 show minimum spacings of beams in order that connections may not interfere.

Fig. 140 shows a beam framing on a skew to another beam. If this bevel from the perpendicular is more than one inch per foot, bent plates should be used rather than angles.

Eccentric connections differ in form with the special conditions of each case but they should be so arranged as to distribute the load, so far as possible, over the whole area of the column section and not entirely on one side.

The foregoing remarks apply also to the design of cast-iron web bases, such as is shown by Figs. 112 and 113. The box of the base should have its metal made to conform in position to the metal of the column and the ribs and base plate should be made of sufficient thickness to form a base stiff enough to distribute the column load uniformly without failure. The tendency in such bases is to split along the line of the central box or across one corner, and the ribs serve to brace the lower plate and resist this tendency. The same tendency would exist in the case of a steel plate riveted to the base of the column and the various shear plates and angles used in such cases are for the purposes of stiffening the plate sufficiently to enable it to distribute the load without failure. The design of such steel and cast-iron bases will be taken up later.

Roof Details. Some of the forms of framing met with in roofs are illustrated by Figs. 141 to 143. If the roof is framed entirely with beams for the purlins and rafters, more simple construction will result if the webs are all placed vertical rather than normal to the plane of the roof. The two forms of connections are illustrated by Figs. 144 and 145. Where the rafters or purlins run over the tops of trusses, however, they are frequently normal

to the plane of the roof and in such cases the connections are generally simpler than where the members frame together.

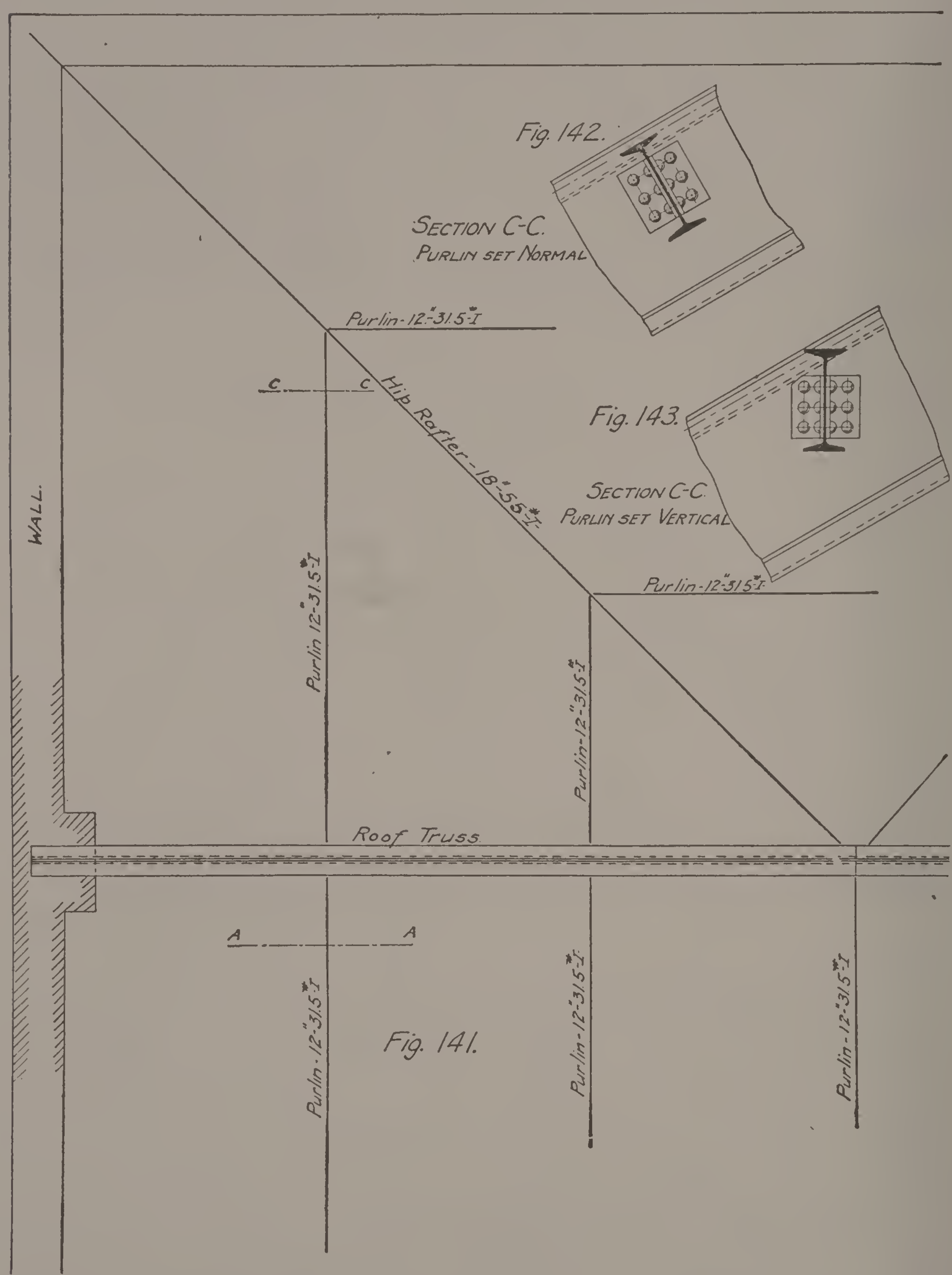


Fig. 141.

Relation to Other Work. It is generally necessary to exercise care in all special connections not to interfere with the architectural features, and to keep the connections within the limits fixed by such features. Full-size sections and details should

generally be worked out, which will determine the exact relation of all portions of the framing to adjacent construction. Such details should be followed in common by the structural draftsman and the draftsman laying out the stonework or interior finish or other adjacent work.

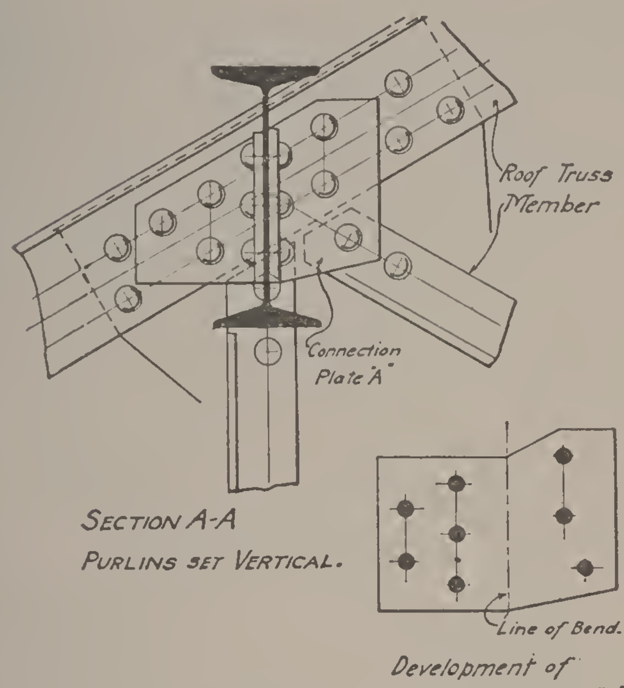


Fig. 144.

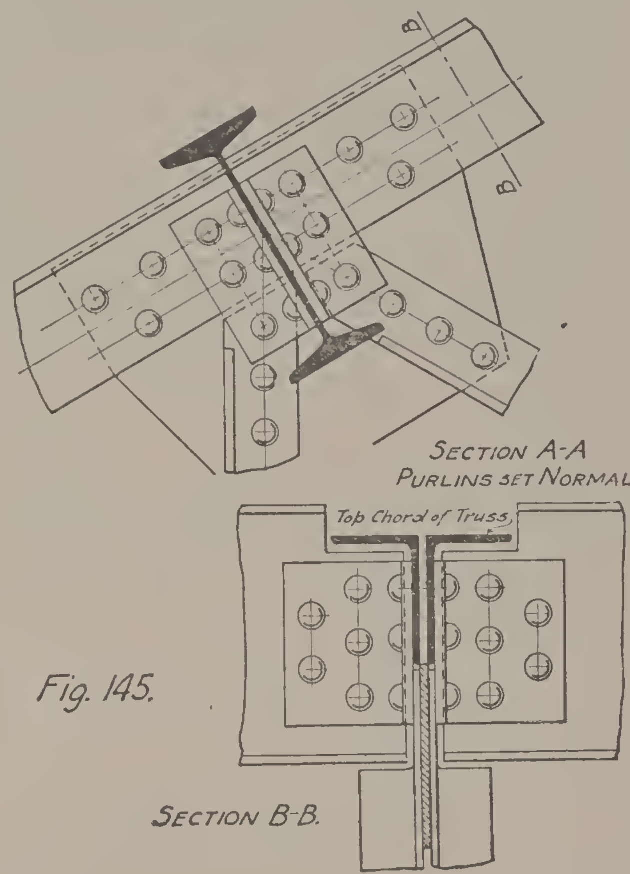


Fig. 145.

Inspection. Steel and iron members are inspected in the mill, the shop, and on the job. As referred to in the section on specifications, the stock from which the material is rolled is systematically tested to determine whether or not it comes up to the requirement of the standard specifications. When it goes into the shop a different kind of inspection is required. First it is necessary to see that the drawings are accurately followed both as regards details and sizes of members and as regards measurements. The rivets and holes must be accurately spaced and the work properly assembled, for if carelessness in such details goes unnoted the different members will not go together when brought to the job and the whole piece may therefore have to be discarded. Secondly, the inspection must cover the quality of the work. This latter division applies almost exclusively to riveted work. Some of the important points to be noted are the following:

The members must be straight and free from twists and bends. Punching must be sharp and true and holes must not be more than $\frac{1}{16}$ inch larger than the diameter of rivet. Holes must not be left with ragged edges after punching. Where necessary to get a clean-cut hole, or where required by the drawings, holes must be reamed after punching.

Members when brought together to be riveted up must have the holes in the different pieces exactly opposite so as not to require drifting in order to bring them together. When driven, rivets must completely fill the holes, and must be of such length that, when the head is formed, the pieces will be brought together under pressure. Rivet heads must be concentric with the axis of the rivet. Column ends or other surfaces specified to be faced must be brought to a true surface exactly at right angles with the axis of the member. All portions of the material not accessible after assembling must be painted before being assembled.

In inspecting cast iron, tests must be made to determine whether or not it comes up to the requirements of the specifications as regards quality. Inspection must also be made to see if the material is free from flaws such as blow holes, pockets of sand and unequal distribution of metal. Where the thickness cannot be measured readily as in the case of columns, small holes are bored to determine this. Where columns are cast in a horizontal position, as they generally are, the tendency is for the core to sag in the center, and therefore it is better to make this test near the center. A sharp blow of a hammer will often indicate unequal distribution of metal. A clear metallic ring indicates a thin shell and a dull heavy sound a thickness of the shell. If the edges are struck with a hammer and pieces fly off under the blow this indicates a brittle texture; a good quality iron should show only a slight indentation. Cast iron should be inspected also for straightness, accurateness of facing of bearing surfaces, and agreement with details. It is better to inspect cast iron before it is painted in order to the more easily discover flaws.

Relation of Engineer to Architect. An essential feature to be observed in all successful designing and detailing by the engineer, is co-operation with the work of the architect. This may seem to the student, at the outset, as a very simple point and

one which will need little special attention. Yet the power to fully and quickly grasp the breadth of the architect's design, and its smallest details as well, and to make the structural design to fully harmonize with his work, will come only by persistent effort.

In some buildings, the work of the engineer, because of the character and purpose of the building, would determine conditions and features to which the architect must conform, but in general the reverse is true. For this reason the burden of harmonizing his work is generally put upon the engineer.

He must see what has been established by the architect and how much he must vary the natural course of his design to conform to these conditions. He must often study long, over what at first seems scarcely possible to accomplish without clashing with the architect's scheme. In the working out of such details and problems, he will need all his originality.

Interpretation of Drawings and Specifications. In preparing the working drawings, the draftsman generally has to do with the design of another. To this extent, therefore, he is not responsible for the harmony of the design with the work of other lines. He is, however, responsible, if such a conflict of design escapes him, for it will be a sure indication that he has not looked at his problem from all sides, and in the light of later and more definite information which was, perhaps, lacking when the design was first made.

In working up the shop details, the draftsman must start with the question constantly in his mind, "How do I know?" He must not fix a measurement, nor establish the position and relation to other parts of a single piece, unless he finds concrete authority in the shape of plans, specifications, or written directions for so doing. Further than this, he must determine that all the information so given is in agreement, for he will be held responsible for failure to discover such disagreements.

There is a great tendency among those young in experience to be guided by what appears to be indicated. Drawings are not always made to exact scale and the structural draftsman should never establish anything by scaling without explicit directions for so doing, and should then make a written record of what has thus been established.

One of the most important instructions which can be given a draftsman, is never to jump at conclusions. Have direct authority for all that is done and be sure your authority is not contradicted in some other place. Oral instructions should be at once written down, as when once followed, they may become a necessary factor in other work. If information is lacking or there is a conflict, however small, in any of the information which is the basis and authority for your work, refer it at once to some one above you who can carry it to the one in authority.

Shop Practice and Use of Detail Shop Drawings. When the shop details are prepared they go first, if the stock list has not previously been made, to the stock department, and a detailed bill of material required in fabrication is made. This is used either to make up the rolling lists or the lists of stock to be taken from the yard. The next step is the making of templates. These are patterns in wood of the exact size and shape of each piece, with the holes located, so that they can be used to mark out the piece itself. Formerly, the template maker did a good deal of the work now done by the draftsman, but in most shops the policy at present is to do as much in the engineering department as possible and to leave nothing to be worked out in the template room or shop.

The templates are sent to the shop and the material goes from one machine to another, being cut to length, coped, mitred, bevelled, sheared and punched as required.

When all the pieces are ready they go to the Assembly Shop and are then riveted up to form the finished piece as required by the drawings. Each piece has its letter or mark to designate it in its passage from the template room to the Assembly Shop; and when the whole piece is assembled it has a mark conforming to what is given on the setting or erection drawing, so that, when received at the job, the erectors will know where it goes.

The final work is the painting, marking, invoicing and weighing and then the shipment.

Relation of Shop Drawings. The basis of all shop details is the setting plan, or erection plan. This shows the framing of the floors and roof, generally a separate plan being required for each floor and one for the roof. This framing plan has all the necessary dimensions to fix the location of each piece, the numbers or marks

designating each piece, the size of piece, and such necessary sections and notes as are required to fix the relation of the different members and to cover any special features.

Each piece must be detailed fully, with cuts, punchings, and framings clearly shown. In general, a standard size beam sheet, column sheet, and girder sheet are used; truss sheets are made to standard sizes as far as possible but on account of the different types and sizes of trusses, more variation is necessary.

Only one tier of beams is put on a single sheet even if of identical detail; also but one section of columns is covered by the same details. If the drawings are going into the mill, a further separation of the different sizes and shapes is necessary so that materials which have to be made in different mills shall not be detailed on the same sheet.

Standard Forms. The specific types of sheets and details will be taken up later.

There are standard forms of connections which cover all but special cases and which are used wherever practicable.

Figs. 146 to 148 show framed, coped, and bevelled beams.

There are certain conventional sizes and standards which should be known to those who have anything to do with working drawings.

A setting plan can be so jumbled and confused by careless arrangement of data, and by poor execution that it will take longer for the man on the job or in the shop to determine its intention than to work out independently what he wants to know. The draftsman should aim to put himself in the place of the shop foreman or erector, who, when he takes up the work, must rely entirely on this plan for all the information. He must aim to give all the necessary information and give it so plainly that it can be quickly seen and cannot be misinterpreted.

Wall lines are shown by red lines in order not to be confused with the beam lines. The walls shown are those upon which the beams rest. For instance, the setting plan of the first floor beams will have the basement walls shown and the second floor plan will have the first story walls shown. Columns are represented by a single line indicating the members composing the columns; this is illustrated by the columns shown in Plate I. It is important to

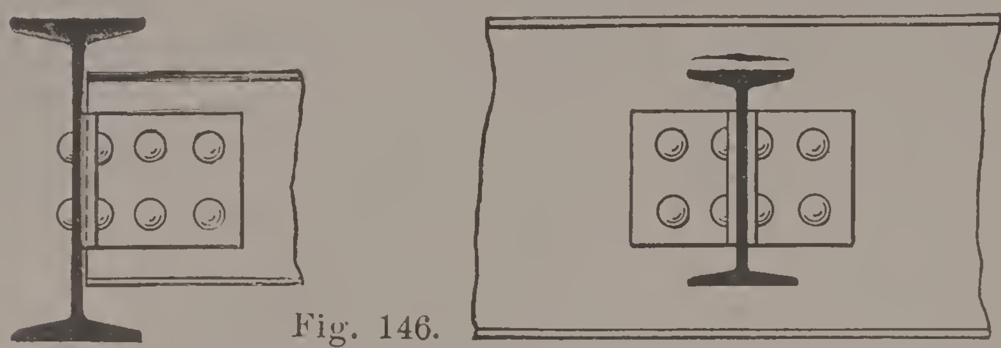


Fig. 146.

BEAM FRAMED BELOW TOP OF GIRDER

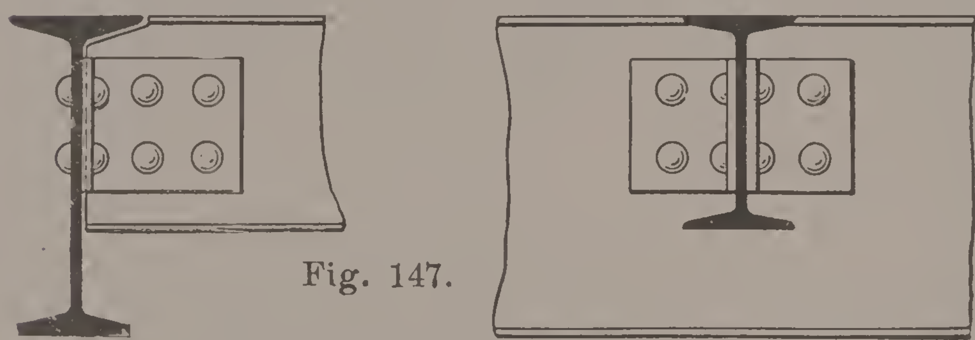
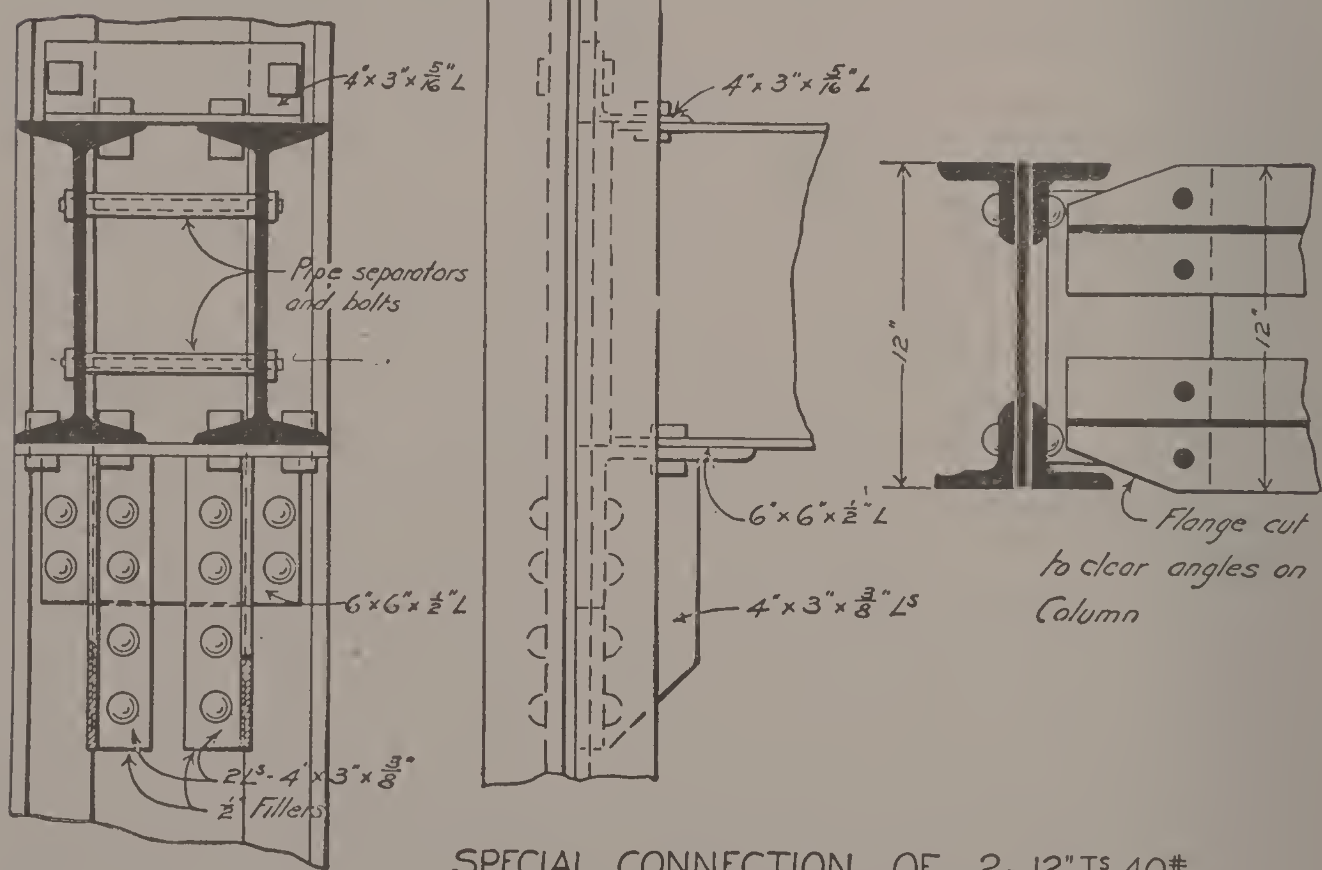


Fig. 147.

BEAM FRAMED FLUSH WITH TOP
OF GIRDER AND COPED TO IT



SPECIAL CONNECTION OF 2-12" Is 40#
TO PLATE AND ANGLE COLUMN
Fig. 148.

indicate clearly the composing elements so as to show which way the web of the column sets.

Beams and girders are indicated by single lines corresponding to the center lines of webs of beams and backs of channels. All lines indicating the steel members should be heavy black lines. Beams framing into a girder or column are indicated by stopping the line of this beam a little short of the line of the girder or of the column. Where a beam runs over another, the lines indicating them cross or, if there is likely to be a question, a note is put on to this effect.

Lintel beams are shown on the framing plan of the floor just above the opening; for instance, lintels over the first story openings would be shown on the second floor plan.

Measurement lines are put on in red, and should locate all bearing walls and all columns and each piece of steel. Beams are located by their center lines; measurements to a channel should go to the back. Channels placed against a masonry wall are generally put with their backs one-half inch away from the wall.

Tie rods are not located by dimensions on the plans except in special cases where a rod must come in a definite position to escape some other member.

The size of beams are marked along the line indicating the beam. In cases where there are a number of beams in the same bay of the same size, it is better to use the symbol "do" or write the size once and indicate on the drawing.

Each piece is given a number. The pieces may be numbered consecutively or it is the practice in some cases to give the same number to all beams which are identical as regards size and detail. In all cases, the number or letter which serves to identify the piece should be put on conspicuously as this is what should be easily seen when using the plan.

The size of bearing plates should be specified either at the wall end of the beam or by a general note, giving the sizes of plates for different sizes of beams.

The general notes should also give the letter designating the floor as "A" for first floor, "B" for second floor, etc.

The grade of underside of all beams should be given in the

body of the plan or by general notes and the relations of tops or bottoms of all beams to each other and to the finished floor line.

Sections should be made showing the framing over windows and of all special connections, and the relation of the different members to each other. In short, the setting plan must be a complete and final expression of all the data which has been gleaned from the general plans and specifications, and must be a guide to the shop man and the man at the job in fabricating, shipping, and putting the frames together.

Beams are generally marked thus: "A-No. 125," or "D-No. 56;" the lowest tier of beams being given the first letter in the alphabet, and so on in order, or First Floor No. 125, Fourth Floor No. 56, and so on.

Columns are generally marked "1st Section No. 10" or "3rd Section No. 5." Columns are sometimes made in only one story lengths but more often in two. They are sometimes marked thus: Col. No. 10 (0–2) or Col. No. 5 (4–6).

The joint in a line of columns should come just above the connection of the floor beams.

Mill or Shop Invoices. These are detailed schedules sent out by the mill when shipments are made. They give the designation of the piece with its weight and all connections and the mill marks, also the marks identifying it on the setting plan. These invoices are valuable as showing just what material has been shipped and in what car and on what date, and also serve to fix the weight when this is made the basis of payment. A form of invoice used by the mills of the Carnegie Steel Company is given by Fig. 149.

Estimating. In making an estimate of the cost of steel work, the basis is always the weight of steel of different kinds. This is determined by taking from the general or framing plans a detailed schedule of each piece of steel. As framing plans are always shown to a small scale and include only the general features of the framing, this work requires special training before it can be done accurately and in the most efficient manner.

In taking off quantities, the estimator generally scales the lengths as these are not usually given by figures. A test of

measurements given by the general plans should be made when possible, to see how nearly to scale the drawings are made. A close estimate should not vary much more than $2\frac{1}{2}\%$ or 3% from the actual weight, so it will be seen that considerable care is necessary.

CARNEGIE STEEL COMPANY.

HOMESTEAD STEEL WORKS.

STATEMENT OF DETAILS OF CONSIGNMENT MADE March 16th, 1902 BILL NUMBER 119437

Brown and Burnham SHEET NUMBER 3595

Boston

Mass

To Same

Boston, Mass IN CAR B and O 74918

VIA B and O Co. W. Va. Co. P and R Co. C. R. R. of N. J. Co. N. Y. N. H. and H.

PURCHASER'S ORDER No	PRICE No	MILL ORDER No	HEAT OR BLOW No	PIECES	SECTION OR SIZE	WEIGHT PER FOOT OR THICKNESS	LENGTH FEET	LENGTH INS	TOTAL LENGTH FEET	TOTAL LENGTH INS	WEIGHT	DESCRIPTION	MARK
	Johnson Bldg.												
			Painted Detroit Graphite One Coat										
				6th Floor									
1/12	CN	646		1	6	12 1/4	7	6½			92	Hd	41
	370	118		4	6x4	x 7/16	0	3			15	Ftgs	
		Bx		4	Rvt	3/4	0	2½			2		

Fig. 149.

Individual estimators have different methods of separating the different classes of material.

The following are the general divisions of material:

I. Beams and channels 15 inches and under.

(*a*). Plain beams and channels.

(*b*). Beams and channels, punched two or more sizes of holes in web.

(*d*). Beams and channels, punched in web and flanges.

(*e*). Framed beams and channels.

(*f*.) Framed and coped beams and channels.

II. Beams and channels 18 inches and above.

The above divisions apply also to these sizes of beams and channels.

There is an extra charge for all beams and channels over 15 inches deep, therefore these sizes must be separated.

Further, all the other shapes must be kept separated from beams and classified by themselves in a manner similar to the

division of beams. For instance, the members composing the columns as plates, channels, angles, zee bars, etc., are each kept by themselves.

All connections of beams to girders and columns are charged at a different price from ordinary angles or plates, and must therefore be figured separately. In a like manner tie rods, anchors, beam plates, column bases, separators and bolts all are classified separately.

It is evident that these different divisions cannot be made at the time the schedule is taken from the plans, and it is customary to take off the material in order as it appears on the plans, and by some system of marking designate the class to which the piece belongs. The separation is then made when the weights are calculated and the quantities are being totaled.

It is also evident that such things as separators, framing, connections, splices, and other details cannot be taken directly from the plans, but must be calculated largely by the judgment of the estimator. He must be able to see just what character of connection is required in order to classify correctly his material as he takes it off.

Effect of Changes. Changes in details must sometimes be made from causes beyond the control of the draftsman. A change in the location of certain members, or the general arrangement at a certain point, may make it necessary to revise drawings already made and perhaps sent to the shop. In such cases, the drawing generally bears the same number and is marked revised. In case additional sheets must be prepared, of course new numbers are given to them. In sending out a revised drawing, instructions should be sent to have the original sheets returned in order that they may all be destroyed and thus remove all liability of the material being made up by the old drawings. Revising details already completed and checked are fruitful sources of errors. Unless the greatest care is exercised, the changes made will affect the relations to some other members and the details of some other portions of the work not at first apparent. The draftsman should have this point always in mind and review all possible connections to other work when revising any details.

Use of Details in the Work. The detail drawings must

frequently be used in determining features of other work and in laying out such work, and for this purpose the detail drawings should contain information enough to establish the relation of the steel to such working lines as finished floor levels, datum line, ashlar line, party line, and such other lines used in the general drawings to establish the relations of the different parts of the work.

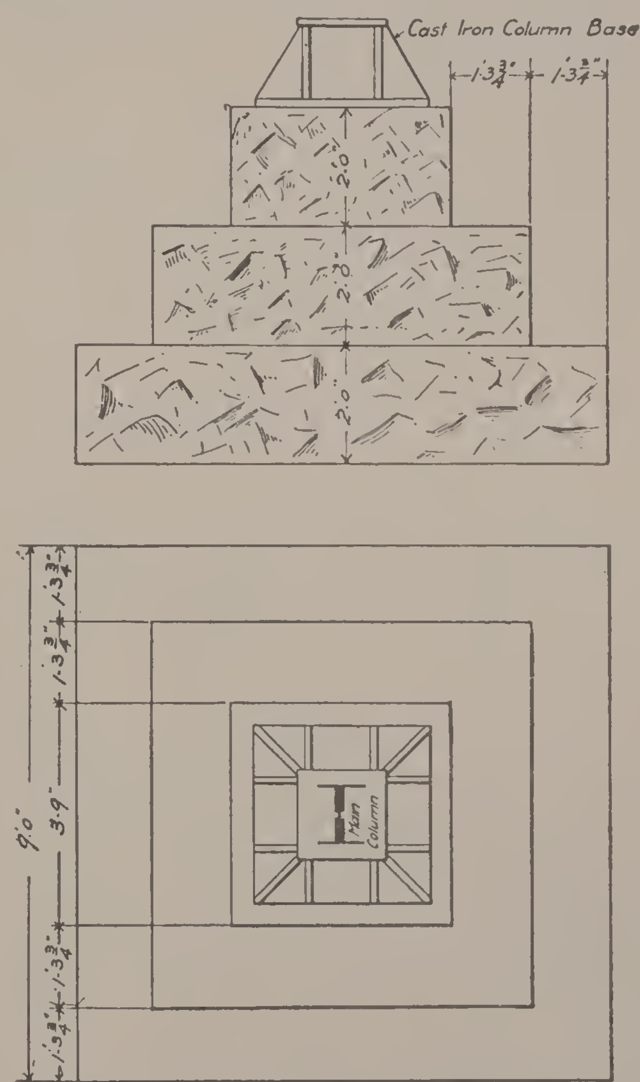
FOUNDATIONS.

There are three general types of foundations.

(1) Spread foundations.
(2) Foundations to bed rock by piers or caissons.
(3) Pile foundations.

The form of foundation used depends largely on the character of underlying soil, and the amount and arrangement of the loads and the depths which can be allowed for foundation.

Spread Foundations. This general division covers all forms of construction in which the foundations are spread out sufficiently, either by offsets of masonry or by steel beam grillage, to distribute the load without exceeding the safe-bearing capacity of the soil. Fig. 150 shows a masonry footing and Fig. 151 a grillage footing. Bearing capacity of soils vary considerably and there are no rigid limits fixing the allowable bearing values of different kinds of soil. Table XIX represents good general practice.



BLOCK STONE FOOTING UNDER COLUMN
Fig. 150.

In some localities, notably Chicago, footings, if they are to be spread, require the use of beams because of the relatively thin bearing stratum, the low allowable bearing value, and the magni-

tude of the loads to be supported. To offset by successive layers of masonry would require too great a depth for the thin layer of hard clay; it thus necessitates the use of grillage beams. In other places either masonry offsets or grillage could be used.

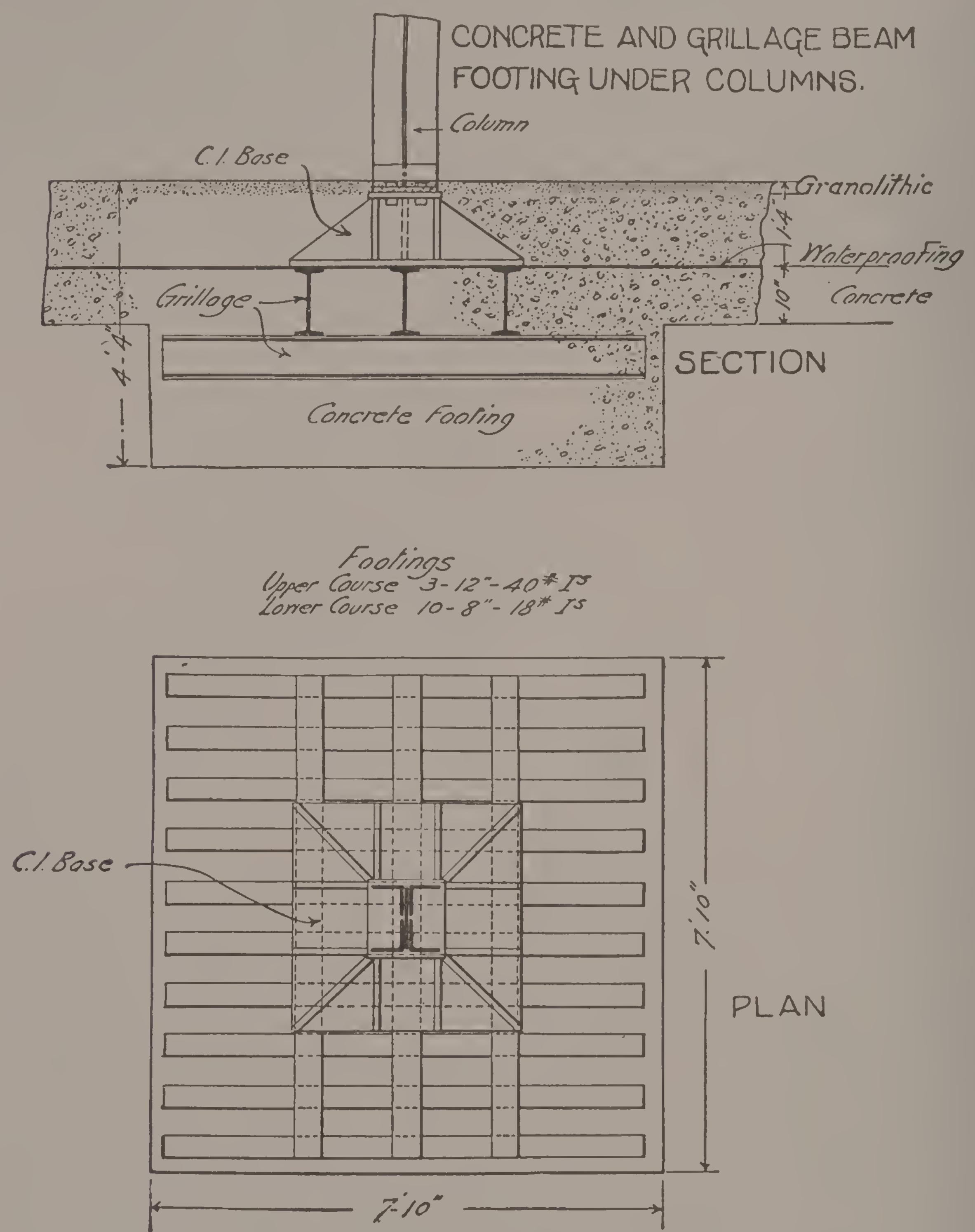


Fig. 151.

In Boston the usual soil encountered is a stiff blue or yellow clay, 15 feet or more thick and underlaid with a boulder clay of varying depth, but generally of from 15 to 75 feet. Under these conditions footings for isolated columns are very commonly made by offsetting the masonry until the required area is gained. In

some cases the water level and a combination of footings may make it desirable to spread by means of beams.

Caisson Foundations. In a yielding soil, or where the area available for spread footings is not sufficient, or where these footings would be excessive in size, foundations are often carried to bed rock.

The most common method is by the use of compressed air. Generally steel caissons, of the size of the pier, are used. These caissons have their edges extending below an air-tight floor, thus forming what is called the working chamber. Compressed air is forced into this chamber which keeps out water and soft material and enables workmen to excavate. The workmen gain access through air-tight shafts with double sets of doors forming an air-lock between the pressure below and above; they of course work under the pressure of the compressed air. The material excavated is hoisted up through shafts and the caisson is sunk by building up the masonry foundation in the caisson at the same time the excavation is going on and this weight sinks it down. When the caisson has reached the grade at which it is to rest, the working chamber is filled with concrete making a solid foundation.

Pile Foundations. Piles support their load both because of the friction between their surface and the surrounding soil and because of resting on solid stratum at the bottom. In some cases probably the greatest support is from the friction on the surface of the piles. They should be driven into a solid stratum far enough to resist any tendency to side deflection. In some instances, notably in old wharf construction, the piles have been driven through a soft mud perhaps fifteen or twenty feet, and only a few feet into the hard clay below. In such cases the piles have deflected under heavy loads, and have assumed an inclined position, their tops having moved laterally ten or twelve feet. This of course causes failure.

Piles should be driven with care so as to be kept in line, and the blows should not be so heavy as to cause brooming either of the head or point. A number of rules are given for driving piles and for determining the load they will support. Two rules in common use are the following:

Baker

$$P = 100 \, [\sqrt{W \, h + (50d)^2} - 50 \, d]$$

W = weight of ram in tons
w = height of fall in feet
d = penetration at last blow in feet
p = pressure in tons to just move pile.

The last blow must be struck on sound wood.

Trautwine

$$P = \frac{46 \text{ W } \sqrt[3]{\text{h}}}{1 + 12 \text{ d}}$$

In determining this last penetration it should be observed that the pile must be driven continuously, as, if allowed to stand some time between blows the soil becomes settled around the pile and the friction thus makes the penetration much less.

Some authorities advocate driving piles with the bark on and some with it off. If the bark is on, the piles should be cut in the fall as otherwise the sap between the bark and wood will ultimately cause the two to separate and the pile to slip within its bark.

The building laws of some cities require the piles to be camped directly with granite levelers; most authorities, however, prefer a thick bed of concrete encasing the heads of the piles and capping them at the same time.

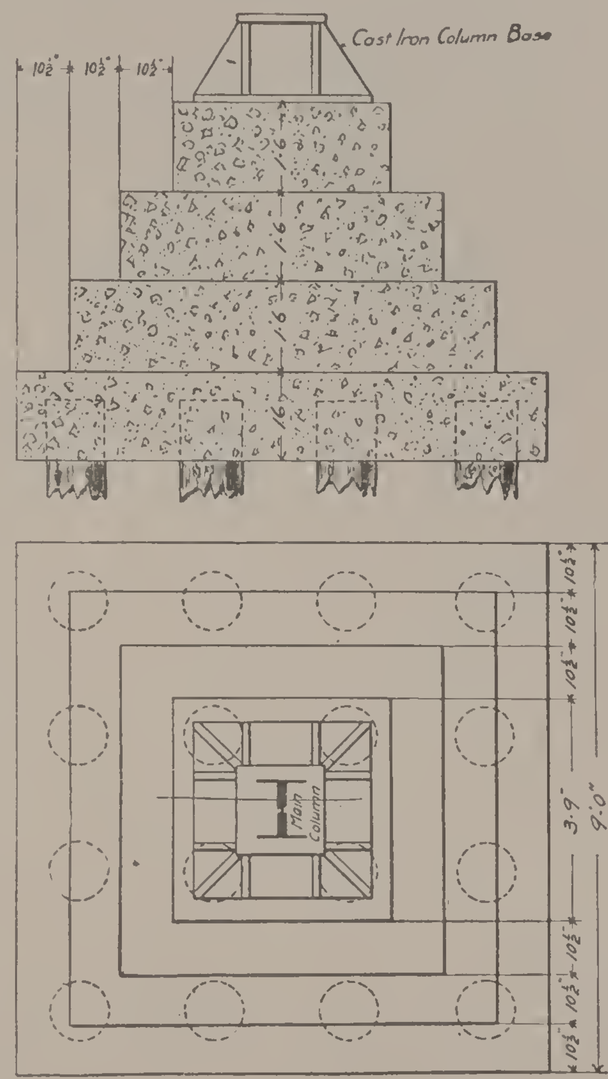
The factor of safety should be from 2 to 12, varying with the accuracy of the knowledge of the loads to be carried and with the closeness with which the formulæ used fits the conditions of the special case. Fig. 152 shows a footing supported by piles.

Fundamental Principles. The essential points in the design of foundations is not to overload the soil so as to cause excessive settlement, and to so arrange and distribute the loads as to cause the settlement to be uniform. Some settlement is practically sure to occur in almost all cases, but unequal settlement causes strains in the structure and cracks in the masonry.

If the supporting power of the soil is nearly uniform over

the whole area of the building, the first problem is to determine the amount of load on each footing. This is not as simple as would at first appear. Not only is it uncertain just how much live load will be carried, but also what proportion of the whole building will be loaded with this live load.

Furthermore the dead load carried by the columns supporting the walls forms a much larger proportion of the total load on these columns than does the dead load carried by the interior columns. The different proportion of loading on the columns must, therefore, be brought to a common basis by some assumption. In the case of office buildings, the actual live load which reaches the foundations is probably a small proportion of the total live load calculated over the whole area of all the floors. Moreover, the building has considerable time to settle from its dead load before any live load comes upon it. In order, therefore, to harmonize the settlement between wall and interior columns it is better to use as a basis the dead loads and a certain percentage of the live loads — say 25 per cent.



CONCRETE AND PILE FOOTING UNDER COLUMN
Fig. 152.

A table should be made of the dead load and 25 per cent of the live load of each column footing. The areas should then be made such that these loads on the soil would be the same per square foot in each case. Care must be exercised that in so doing, the total load of dead and live, or if the building laws under which the work is done permit of a reduction in live load, that this percentage of live and dead does not bring the load per square foot above the specified amount. In general, this will not be the case if the column footing, in which the proportion of dead plus 25 per cent live to the total load is the least, is first proportioned for total load

and the others then made proportional to it. The following example will illustrate this point.

Problem. Suppose columns as follows :

No. 1 Dead+25% live=407,000. Total load=629,000
No. 2 Dead+25% live=190,000. Total load=245,000
No. 3 Dead+25% live=275,000. Total load=465,000

Maximum allowable bearing on soil from total load to be 5,000 pounds per square foot.

In No. 1 the dead+25% live is 64.5% of the total load on this column.
In No. 2 the dead+25% live is 80 % of the total load on this column.
In No. 3 the dead+25% live is 59.2% of the total load on this column.

If then, we take column No. 3 as the basis we have the required area equal to 465,000 divided by 5,000 or 93 square feet. This gives 2,960 pounds per square foot from the dead + 25% live load.

For No. 1 in order to have the pressure from the dead + 25% live the same as in No. 3 we shall require 407,000 divided by 2,960 or 137.5 square feet. This area gives 4,560 pounds per square foot pressure from the total load.

In column No. 2 we have 196,000 divided by 2,960 or 66 square feet required, and the pressure from the total load is 3,700 pounds per square foot.

A further provision which must be made is to bring the center of gravity of the resisting area, or loaded area, coincident with the axis of the load. The same principle of a strut eccentrically loaded applies to a footing in which eccentricity of loading exists. In such a case equal distribution on the soil is impossible as the side on which eccentricity exists will always be loaded the most. Furthermore, a bending moment, as in a strut similarly loaded, will occur in the foundations, and even a slight eccentricity, if the load is considerable, will cause heavy strains in the footing. This latter point is sometimes difficult to accomplish because of the restricted area available for the footings. In some cases the loading and bearing capacity make it necessary to combine the footings of several columns, or the necessity of combining the foundations under an old wall with new footings, or

of providing for a future wall or column on the same footing, or of keeping the footing for a column in a party wall entirely within the party line,—any or all of these conditions may make it impossible to fulfil exactly the conditions previously mentioned. Departure from these principles should be as slight as possible, and when necessary direct provision should be made for the additional strains consequent thereon.

The necessity of keeping footings inside of party lines, and the desire to make the axis of load conform to the center of gravity of area, sometimes results in the use of cantilever construction. These cantilevers are in some cases laid directly over the beams forming the grillage in the footing. This construction makes the actual point of application of the loads uncertain as any deflection would tend to throw the load on the outer beams. A better construction is the use of a shoe with a pin bearing.

Improvement of Bearing Power. The supporting power of all soils is improved by compacting, by mixing sand or gravel or by driving piles which prevent the spreading of the soil as well as compacting it. Drainage of a wet soil also greatly improves its bearing power.

The following table taken from Baker's "Treatise on Masonry Construction," gives values for general use in determining the bearing power of soils:

TABLE XIX.

	Safe Bearing Power Tons per Sq. Foot.
Clay in thick beds, always dry	4 to 6
Clay in thick beds, moderately dry	2 to 4
Clay soft	1 to 2
Gravel and Coarse Sand, well cemented	8 to 10
Sand compact and well cemented	4 to 6
Sand clean and dry	2 to 4
Quicksand, alluvial soils, etc	½ to 1

The bearing power of clay depends largely upon the degree of moisture.

Foundations on clay, containing much water, and undrained, are liable to settlements from the escape of the water either by adjacent excavations, or by the squeezing out of the water.

Moist clay in inclined strata is liable to slide when loaded. Clay mixed with sand or gravel will bear more load than pure clay. Sand will bear more load than ordinary clay, and when in beds of sufficient thickness and extent to prevent running, will bear heavy loads with little settlement. Sand sufficiently fluid to run, as quicksand, cannot be easily employed to carry foundations.

Grillage Foundations. The simple grillage foundation is illustrated by Fig. 151. The method of calculating the beams

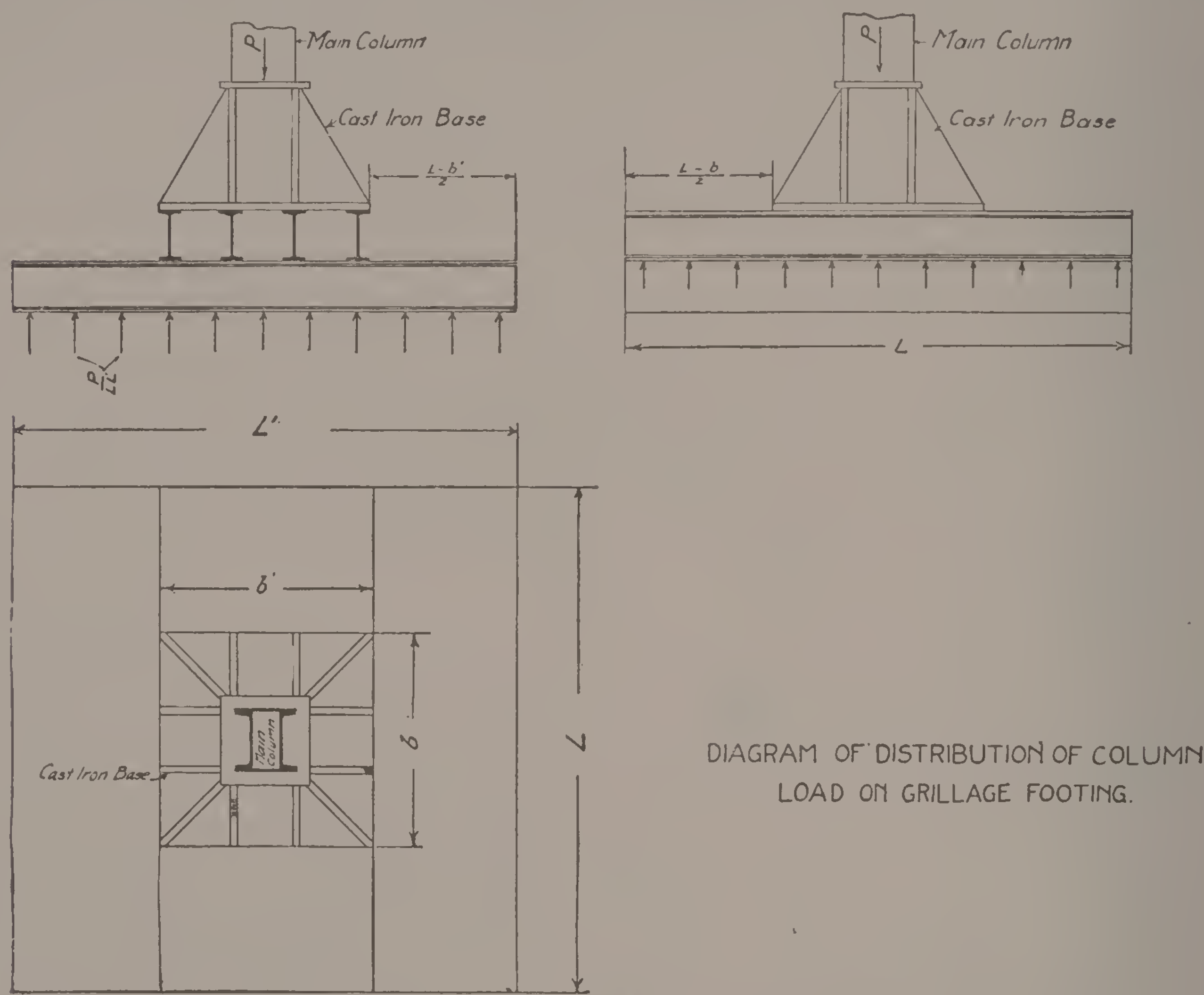


Fig. 153.

composing the grillage involves assumptions as to the conditions of distribution of loading and stresses. One method is given in Cambria, Page 263. This method involves the assumption that the beams can deflect from the line of axis of column. Such a condition, however, would lead to the cast-iron base bearing at its outer edges only; this would involve strains for which these bases are rarely designed. Another assumption and one more in harmony with the assumption of the ordinary beam theory, is that the beams of the upper tier are fixed for the portion under the column base. Under this assumption the load is distributed uni-

formly by the upper tier and the stress in the free portion is calculated by the formula for a beam fixed at one end and free at the other.

Referring to Fig. 153; suppose the column load is P, and by the principles already given the extreme dimensions of footing are L and L′ in feet. The length of the beams in the two tiers can be taken as L and L′ also. Then if b and b' are the dimensions in feet of the column base, and the beams in the upper tier are placed the same width out to out of flanges as the column base, $\frac{L-b}{2}$ = projection of the upper tier, and $\frac{L'-b'}{2}$ = projection of the lower tier. The load per square foot on the upper tier is $\frac{P}{b'\ L}$, and on the lower tier is $\frac{P}{L\ L'}$. The moment in inch pounds,

therefore, is $M = \frac{1}{2} \times \frac{b'\ P}{b'\ L} \times \frac{(L-b)^2}{2} \times 12$

$$= \tfrac{3}{2} \times \frac{P}{L}(L-b)^2 \text{ for the upper tier}$$

$$\text{and } M' = \tfrac{1}{2} \times \frac{PL}{LL'} \times \frac{(L'-b')^2}{2} \times 12$$

$$= \tfrac{3}{2} \times \frac{P}{L'}(L'-b')^2 \text{ for the lower tier.}$$

These formulas give the total moment borne by all the beams in the tier. The number of beams is generally determined by the dimensions of the footing, the beams of the upper tier being placed with their flanges generally not much more than 6 inches apart in the clear, and those of the lower tier from 6 inches to 12 inches. The number of beams being determined, the moment each bears is obtained by dividing the total moment by the number of beams; and by dividing this individual moment by the allowable fibre stress the required moment of resistance and hence the size of beams is obtained. Since the concrete and steel act together, a higher fibre strain can be safely allowed; this should in general be not more than 20,000 pounds per square inch, however.

Some trial and reproportioning of dimensions may sometimes be necessary to keep within the limits of depth and number of

beams desired. Grillage beams in foundations should have the concrete thoroughly tamped around them, and it is preferable that the steel should be coated with neat cement instead of a coat of paint.

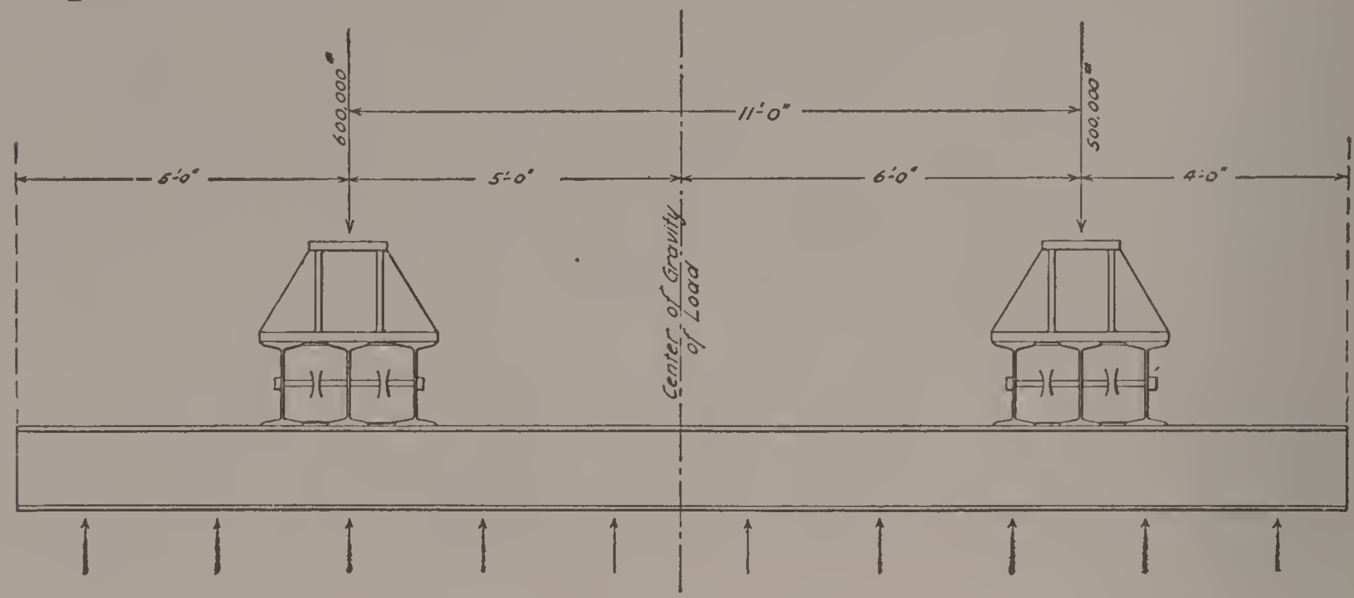


DIAGRAM of GRILLAGE FOOTING.
Fig. 154

The following problem will illustrate the method of procedure in the case of combined footings.

Suppose two columns loaded and spaced as shown by Fig. 154, and let the allowable bearing on soil be 5,000 pounds per square foot. Let the dimensions of the footing be 20′—0″ ×11′—0″=220 square feet. The determination of the size of base is largely a matter of judgment and depends upon the amount of load and the degree of spreading necessary to keep the size of grillage beams, or masonry offsets, within the limits which are economical. Suppose in this case the base is 3′—6″ × 3′—6.″ The load per square foot in the upper tier is therefore $\frac{1,100,000}{20 \times 3.5} =$ 15,714 pounds. The moment on this tier will be a maximum either at one of the columns or at some point between them. The center of gravity of load is $\frac{600,000 \times 11}{1,100,000} = 6$, or 6 feet from the lighter load. This fixes the projection of the footing beyond the loads as 4 feet from the light load and 5 feet from the heavy load. The beams between the column loads are in the condition of a beam fixed at the ends and loaded with a uniformly distributed load. The moment may therefore be taken as approximately $\frac{2}{3}$ of

that for a beam simply supported. The moment between the columns will be a maximum where the shear is zero. To determine this start from one end, say the left-hand end, and determine the distance to the point of no shear by dividing the concentrated load by the load per linear foot; $\frac{600,000}{55,000}=10.9$. If the load is assumed uniformly distributed over the upper tier the greatest moment outside of the column load will be at the end having the greatest free length. The maximum moment therefore in this case will be at the edge of the base plate of the column at the left-hand end or 10.9 feet from this end. Call these moments M and M′ respectively.

$$
\begin{aligned}
M &= \tfrac{1}{2} \times 55,000 \times 3.25 \times 3{\cdot}25 \times 12 \\
&= 3,487,000 \text{ inch pounds} \\
\text{and } M' &= \tfrac{2}{3} \times [55,000 \times 10.9 \times 5.45 - 600,000 \times 5.9] \times 12 \\
&= 2,181,800 \text{ inch pounds.}
\end{aligned}
$$

If the allowable fibre strain is taken at 18,000 pounds per square inch, the required moment of resistance $= \frac{3,487,000}{18,000} =$ 194.

The offsets in masonry footings can be determined by the formula for a beam fixed at one end and loaded uniformly. A general practice and one in fairly close accord with the results of the above formula is to draw lines at 60 degrees with the horizontal from the edges of the column bases and where these cross the joint lines (the thickness of the courses having been assumed) will be the vertical face of the course. When the structure is of such a character that wind load affects the foundations, this must be considered in addition to the other live loads. Such cases would be narrow and very high buildings, chimneys, monuments, etc.

While the concrete and imbedded steel beams in a footing are undoubtedly much stronger than the simple beams, it is not customary to figure the beams in such cases by the theory applying to steel imbedded simply in the tension side of concrete. Footings of this character are employed sometimes and their design will be taken up later.

Cantilever Foundations. The case of cantilever construction supporting a party wall is illustrated in Fig. 155. Let P be the wall column load, a the distance in feet from wall column to the pin bearing forming the fulcrum, and b the distance in feet from

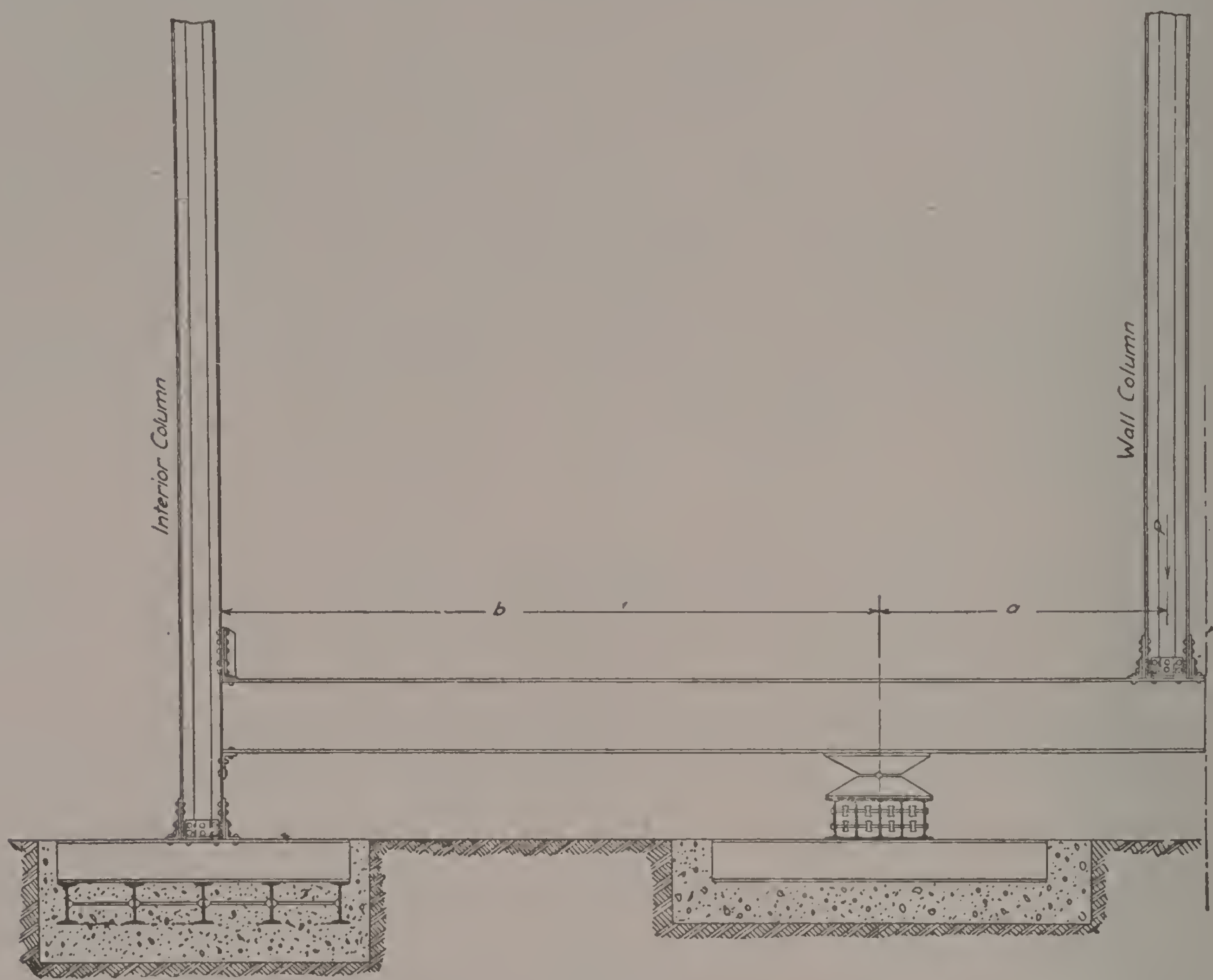
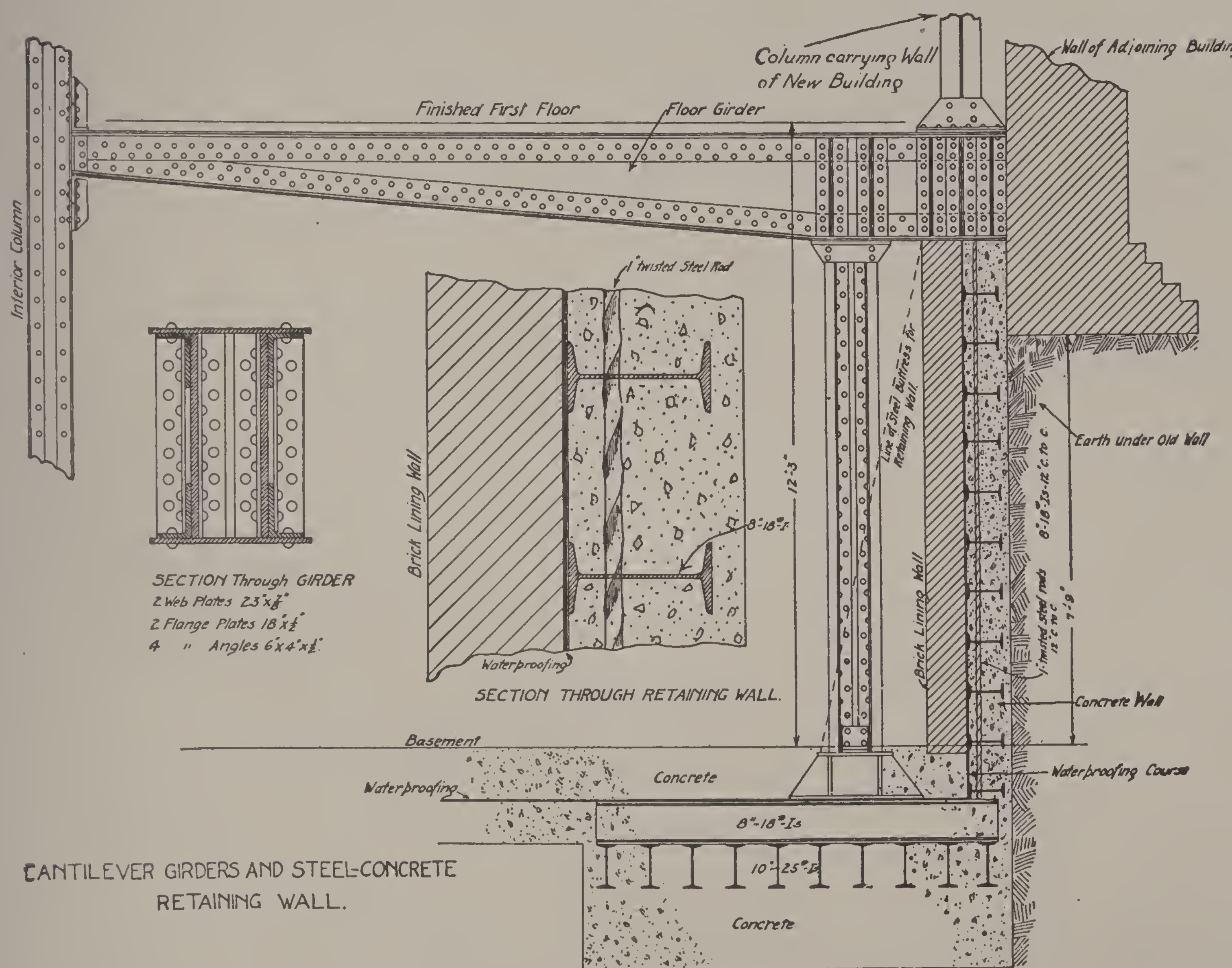


Fig. 155.

fulcrum to column at opposite end of cantilever. Then the load on fulcrum is $\frac{P(a+b)}{b}$. The distance a should be taken so that the fulcrum can be at the center of the footing and still keep within the party lines. Sometimes this cannot be done, and then the footing has to be designed to take account of this eccentricity of bearing. The cantilever is designed by determining the maximum moment and shear. The maximum moment in the above case is at the fulcrum and is Pa in foot pounds. In case the girder is a riveted girder, as is often the case, other features must be considered in its design, as will be explained later.

In case the cantilever is in the floor, as it sometimes is, as shown by Fig. 156, and in addition to the wall column, carries a floor load, then the position of maximum moment must be deter-

mined in a manner similar to that explained for combined footings. The connection of the cantilever at the interior column must be designed to resist this upward tendency and in case the reaction from the dead-wall load is greater than the dead load carried by this column the cantilever arm should be extended to the next column so as to decrease this reaction ; or the column must be anchored and all connections designed to resist this upward reaction.

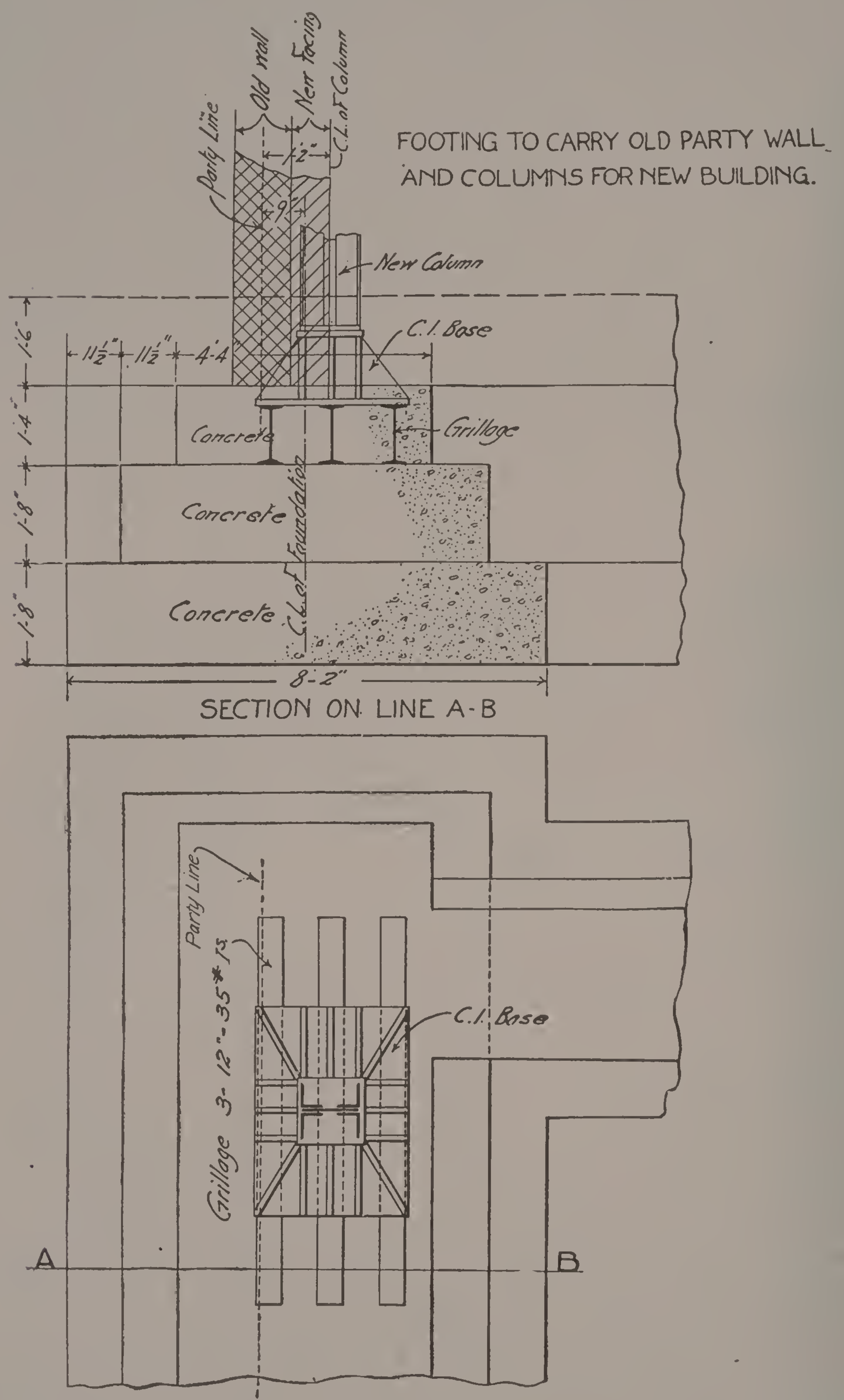


CANTILEVER GIRDERS AND STEEL-CONCRETE RETAINING WALL.

Fig. 156.

Fig. 156 shows also a steel concrete retaining wall to hold up the earth under an adjoining building which foots some distance above the new foundations.

Fig. 157 illustrates the case of a party wall foundation designed to carry a future wall column for the adjoining building and the column of the present building. The eccentricity of bearing is shown in this case, and this and the necessity of spreading in the direction of the wall rather than across it are the important features.



PLAN OF FOOTINGS AT CORNER
UNDER CORNER COLUMN AND OLD WALL.

Fig. 157.

The matter of design of foundations is one always requiring accurate knowledge of the special conditions incident to the problem and the nature of the soil, and is largely influenced by practical considerations and the judgment of the designer. It is not safe to lay down any fixed values to be followed in all cases. Foundations in soil which are at all questionable, should never be designed except by an expert, who is capable of judging the extent to which the ordinary methods of procedure must be modified.

Retaining Walls are walls built to resist the thrust of earth pressure. These walls may also be bearing walls for loads above. The pressure of earth tends to cause failure of the wall in the following ways:

(1). To slide on its base.
(2). To slide on some horizontal joint.
(3). To overturn bodily.
(4). To fail by buckling.

To resist the tendency to slide on its base, the dead weight of the wall, or of the wall and the load it carries, must be sufficient to resist the horizontal pressure without exceeding the coefficient of friction between the material of the wall and the surface upon which it rests.

To resist the second tendency the weight above any joint must be sufficient to resist the pressure above the joint without exceeding the coefficient of friction of masonry upon masonry.

The overturning moment of the earth pressure about the edge of rotation must be balanced by the moment of the weight of the wall and of the superimposed load about the same edge.

The fourth condition applies only to retaining walls supported at their tops and built generally of concrete and steel. A retaining wall so supported would have to resist tension in one side and, as a masonry joint is not intended to resist tension, such construction involves the use of steel. Such construction is becoming more common on account of the saving in space due to the thinness of the wall. In Fig. 156 is shown such a wall. The tensile strength is supplied by the beams running horizontally and the twisted vertical rods.

The resulting pressure due to the thrust of the earth and the

weight of wall and superimposed load must fall within the base in order to give equilibrium, and within the middle third of the base to avoid tension on the masonry joints. Figs. 158–159 show types of retaining walls.

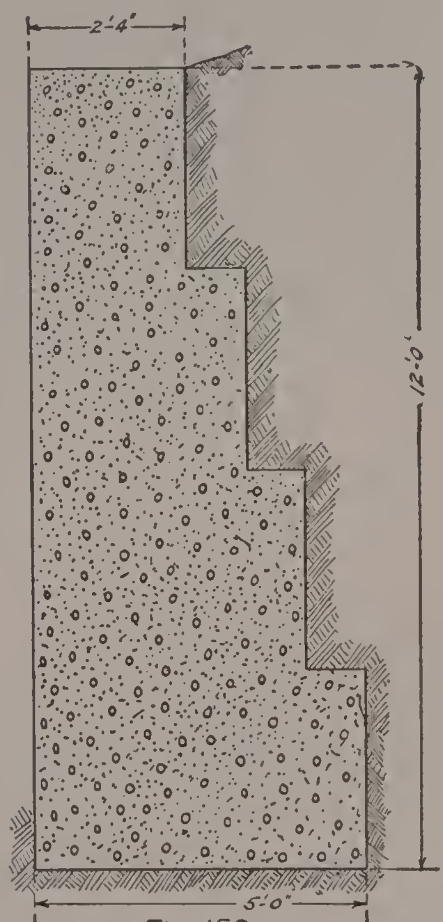


Fig. 158.

TYPES of CONCRETE RETAINING WALLS.

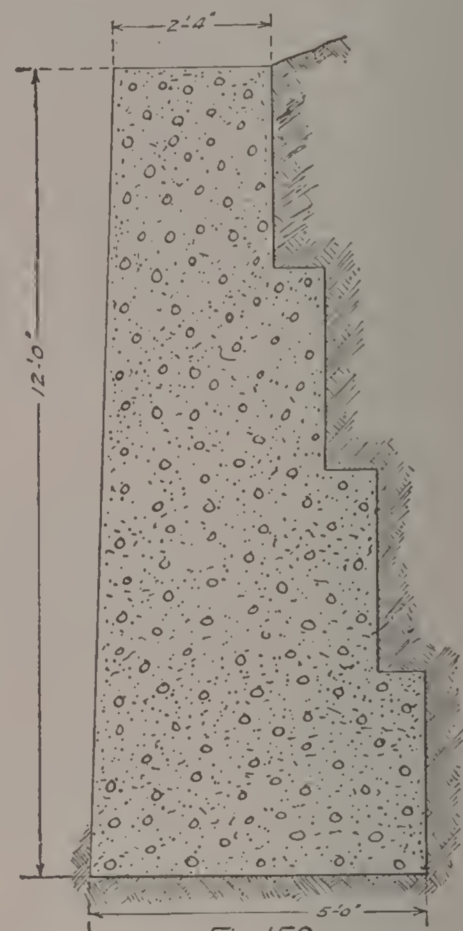


Fig. 159.

Underpinning Shoring and Sheath Piling. Underpinning is the term given to the processes of carrying down old foundations or walls adjacent to new construction to the level of the new construction.

It very often happens that footings of new buildings will be twenty or thirty feet below the bottom of the footings of the walls of an adjacent old building. To leave the old footings at this higher level after the excavation of the new building is made, would necessitate making the wall heavy enough to act as a retaining wall, to resist the pressure on the soil back of it. It is generally more practicable, therefore, to hold up the old wall temporarily by timber braces, needles, wedges, etc., and build new work up under it from the level of the footings of the new buildings. This new foundation under the old wall is called underpinning, and the construction necessary to hold it in place, during the process of underpinning, is called shoring. This latter

term applies to all bracing of old walls or adjacent construction during the construction of the new work, whether the wall is underpinned or not.

Where a mass of earth is to be held in place to enable new excavation to be made without disturbing it, heavy planks set edge to edge are driven down as the excavation proceeds, and braced at intervals by breast pieces or heavy timbers to keep the plank from bulging under the pressure of the earth. This construction is called sheath piling. The planks, generally, are pulled out after the wall, which is designed to permanently hold the earth in place, is built; sometimes, however, it is left in place.

HIGH BUILDING CONSTRUCTION.

Origin of the Types. Iron has been employed extensively in buildings for many years. The first building in this country of what is now known as the skeleton type of construction, was the Home Fire Insurance Company Building, built in Chicago in 1883, of which Mr. W. L. B. Jenney was the architect.

As this was an epoch-making event, it is important to know a few of the details of this building. In an account published in The Engineering Record of January 6, 1894, Mr. Jenney says: "The problem presented by them was to so arrange the openings that all stories above the second or bank floor could be divided to give the maximum number of small offices — say about 12 feet in width — each with its windows conveniently placed and sufficient to abundantly light the entire room. The work was planned quite satisfactorily, but the calculations showed that a material with very much higher crushing strength than brick was necessary for the piers. Iron naturally suggested itself, and an iron column was placed inside of each pier." The chief departure was in making the columns bear all the loads, the walls between the piers supporting only their dead weight for a single story in height. Mr. Jenney states that the difficulty which was feared from the expansion and contraction of the iron columns led to the supporting of the walls and floors independently on the columns. The columns were of cast iron of box section, and the walls were supported on cast-iron box lintels, resting on brackets on the

columns. The floor loads were carried by iron beams, although a few Bessemer steel beams were used, these being the first to be used in this country.

Since the connections were by bolts, the beams were connected together by a bar running through the cast-iron columns, in order to secure a more rigid frame.

The chief advance from that day is in the substitution of steel for all members in high-building construction, and in the development of details in the connections of the members.

Types in Use. There are three main types of high buildings:

1. The class in which the exterior walls are self-supporting, and are designed also to support the ends of the girders carrying the floors. The floor loads inside the walls are carried by steel beams and girders framed between steel or cast-iron columns.

2. In the second class, the exterior walls are self-supporting but the wall ends of floor girders are carried by steel girders and columns.

3. In the third class, the steel frame is a complete unit in itself, and carries all floor loads, and, also, the load of the walls themselves. This latter is the pure skeleton type and the more common form of construction.

Effect on Foundations. The different types have an important effect on the design of the foundations, and in some cases fix their character.

In the first type, the benefit of isolated columns with independent foundations is largely lost, as unequal settlements in the walls themselves and in the walls and columns are likely to result.

In the second type, as all loads are carried on columns which have isolated footings, more equal settlement will probably result, and in the event of the walls settling unequally with respect to the columns, would not affect the steel frame.

In the third class all foundations are generally in effect of the character of isolated piers which can be proportioned to give nearly uniform settlements.

When a party wall makes it desirable to keep all foundations inside of the building by means of a cantilever construction it

can be more readily done in buildings of the third class than in any other type.

Effect of Wind Pressure. Probably the most distinctive problem in high-building construction is the provision for lateral strains in the framework, due to wind pressure. The amount of these strains varies, of course, with the relation of the height of the building to the dimensions of its base and to its exposure on different sides. In the earlier designs, much more complete provision was made for such strains than is now the practice. The laws of some cities, Chicago and Boston for instance, now limit the height to about 125 feet above the street. In other cities, notably New York, buildings of 350 feet or more are allowed. In New York, in buildings having an exposed height of four times or less the least dimension of the base of the building, no special consideration of wind strains is proscribed.

In buildings where the walls are of solid masonry construction and of moderate height, it is not necessary to consider the effect of wind pressure, as the dead weight of the masonry and the stiffness afforded by cross walls and partitions are sufficient to resist the effect of the wind, under ordinary conditions. With the light steel skeleton buildings carried to the height of the modern buildings, the elasticity of the steel frame makes it necessary, under certain conditions, to consider wind pressure. The walls being merely thin coverings, and the partitions also thin and not bonded to the walls, it is apparent that the frame itself must provide all the resistance.

The effect of wind blowing against the exposed surface of a building is

(1) To produce an overturning moment tending to tip the whole building over,

(2) To shear off the connections of the columns to each other, and to cause the floors to slide horizontally,

(3) To slide the whole building horizontally on its foundation,

(4) To twist or distort the frame.

In buildings of usual proportions of height to base, the dead weight, even in the skeleton type, is sufficient to resist a bodily overturning. Some buildings have been built, however, that are

almost of the character of towers or monuments, where this effect must be considered, and provision made for it, by anchoring to the foundations. The action under such conditions will be understood by referring to Fig. 160 which shows the outline of a narrow building, having columns only in the walls. The building would tend to tip about the side opposite to that upon which the wind is blowing, and the columns on the wind side would be in tension, due to the action of the wind. If the load on these columns due to the weight of construction and a small percentage of the live load, to cover weight of fixtures in the old buildings, were less than this tension, the difference would constitute the strain on the anchorage. If the building were safe against overturning, it would ordinarily be safe against sliding bodily, as will be seen from the following consideration:

Suppose a = the width of base
h = the height above ground
p = the wind pressure per square foot
w = the dead weight necessary to resist overturning
f = the allowable coefficient of friction on the foundations
b = length of building

Then assuming the whole surface acted upon by the wind, and the weight of the building acting through its center of gravity

$$w = \frac{p\,b\,h^2}{a}$$

In order, therefore, for the building of the above weight to slide

$$f\,w = p\,b\,h$$

$$f = \frac{p\,b\,h\,a}{p\,b\,h^2} = \frac{a}{h}$$

As the allowable coefficient can safely be taken at .40 this means that for the sliding tendency to be considered the width of base must be .40 or more of the height.

Buildings in which the overturning effect would need to be

TYPES OF WIND BRACING

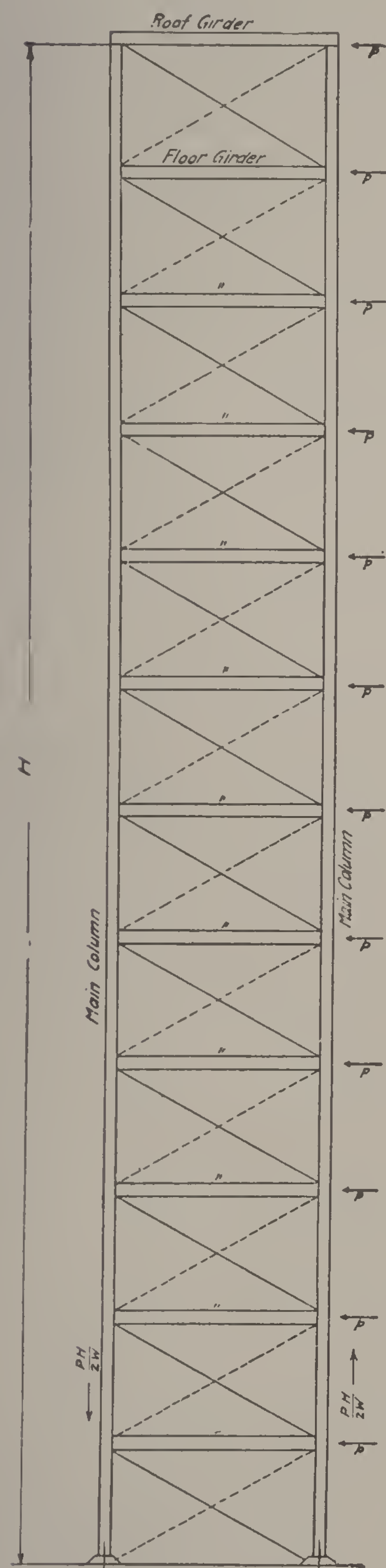


Diagram of one Column Bay Braced with rods to resist Wind Pressure.

Fig. 160.

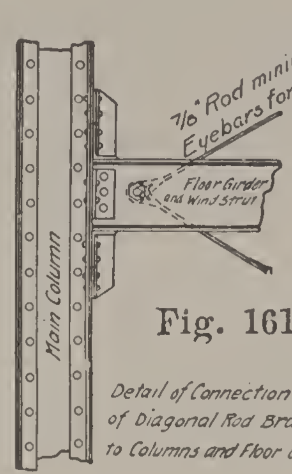


Fig. 161.

Detail of Connection of Diagonal Rod Bracing to Columns and Floor Girders

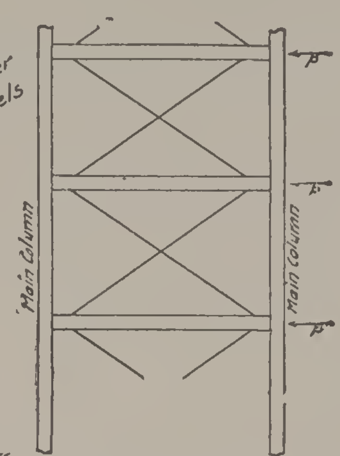
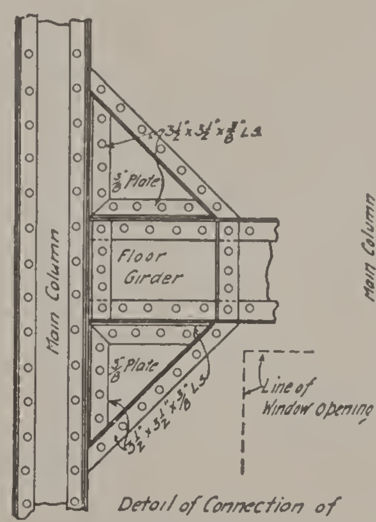


Diagram of one Column Bay Braced by Diagonal Rods to resist Wind Pressure



Detail of Connection of Knee Braces to Columns and Girders

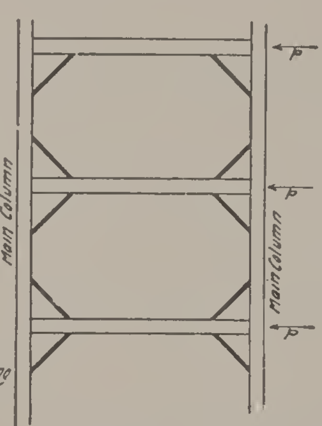
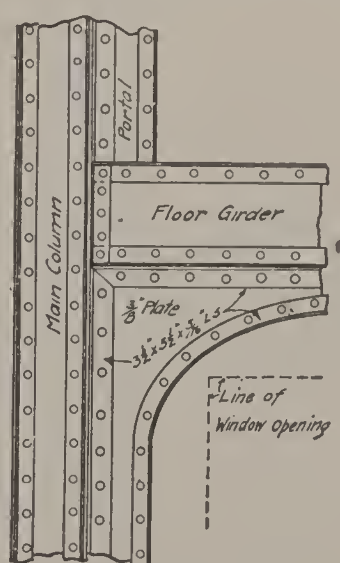


Fig. 162.

Diagram of one Column Bay Braced by Knee Plates and Angles to resist Wind Pressure



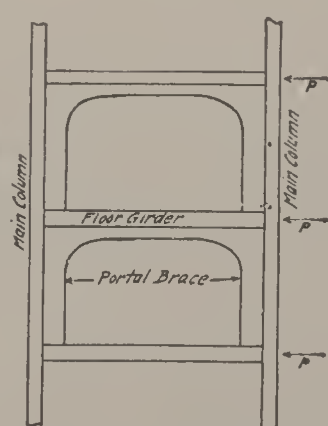


Diagram of one Column Bay Braced by Portals of Plates and Angles to resist Wind Pressure.

Fig. 163.

considered would have a base much narrower than .40 the height so that it is safe to say a narrow building, if safe against overturning, would be safe against sliding.

A further point in this connection is, that ordinarily, the columns do not stop at the ground level, but extend below and therefore have the resistance of the adjacent ground against sliding.

The tendency to shear the connections, and to twist and distort the frame, are ordinarily the most important features of wind pressure and these effects are always present in a high building exposed to wind. The connections necessary for framing the floors and columns may sometimes of themselves be sufficient to provide for these strains ; in other cases special provision must be made.

Wind Bracing. Where special provision has been made it has generally been by vertical bracing between columns, either in the form of diagonal members, similar to the web members of a truss, or by portal bracing in the form of a stiffened plate arched between columns, or by knee braces between the columns and the horizontal members. A modification of the two latter forms has of late years resulted in using a deep girder at the floor levels, in the walls between columns. These different types of bracing are illustrated by Figs. 160 to 163. Their calculation will be considered later. There is always some vibration in high buildings exposed to a severe wind, as has been shown by plumb lines hung in shafts from the top of the building.

The wall covering being carried by the steel frame has greatly changed the methods of erecting a building. Now, the frame is carried up a number of stories, perhaps to its full height, before any work on the walls is commenced. It may then be started at the sixth floor just as well as at the first. The frame is also used as anchorage for the derricks used in erection. The designer or draftsman has, perhaps, little to do with the methods used in erection, but a thorough knowledge of the conditions and general practice which prevails should enable him to arrange the framing so as to facilitate and aid in the rapidity of the erection.

It is not often that a complete system of diagonal braces can be used in the exterior walls, on account of interfering with the

window openings; they are sometimes introduced in the interior walls or partitions. Portal bracing while formerly used to some extent is but little used now. Knee braces and deep stiff girders or struts at the floor levels, are the more common types of bracing. Portal braces, while forming a rigid frame without interfering with the openings in walls, have the disadvantage of being difficult of erection, expensive, and they induce heavy bending strains in the portal itself and in the columns.

Fig. 164 shows the Penn Mutual Building of Boston, during construction, of which Messrs. F. C. Roberts & Co., and Mr. Edgar V. Seeler of Philadelphia were the architects and engineers. This photograph shows the deep girders at each floor level which serve not only to carry the loads but as wind bracing.

The student should also notice the method of supporting staging independently from any floor, and the masonry supported independently at each floor, as shown at the fourth floor.

Figs. 165, 166, and 167 give interior views of the same building. The floor system was put in by the Eastern Expanded Metal Co. and consisted, in general, of a slab 7 inches thick re-enforced continuously at the bottom by 3-inch No. 10 expanded metal, and also at the top for about four feet from the ends. There were also $\frac{1}{2}$-inch round rods bent over the tops of the girders and running down to the bottom of the slab at the center; these rods were used every six inches.

The span of these floor slabs is 17′ — 6.″

These views show also the method of wrapping the columns and flanges of beams with metal lath and plastering.

The student should note, also, the appearance of the centering shown by Figs. 166 and 167, and of the concrete where the centers are removed; the grain of the wood is shown clearly marked in the concrete.

Fig. 168 shows the Oliver Building, Boston, during construction, of which Mr. Paul Starrett was the architect.

This photograph shows clearly the practice of leaving the masonry down for one or more stories and building the stories above. It also shows the iron fascias set in place in the upper stories; this is done in advance of the masonry so that the masonry will fit more accurately and neatly around them.

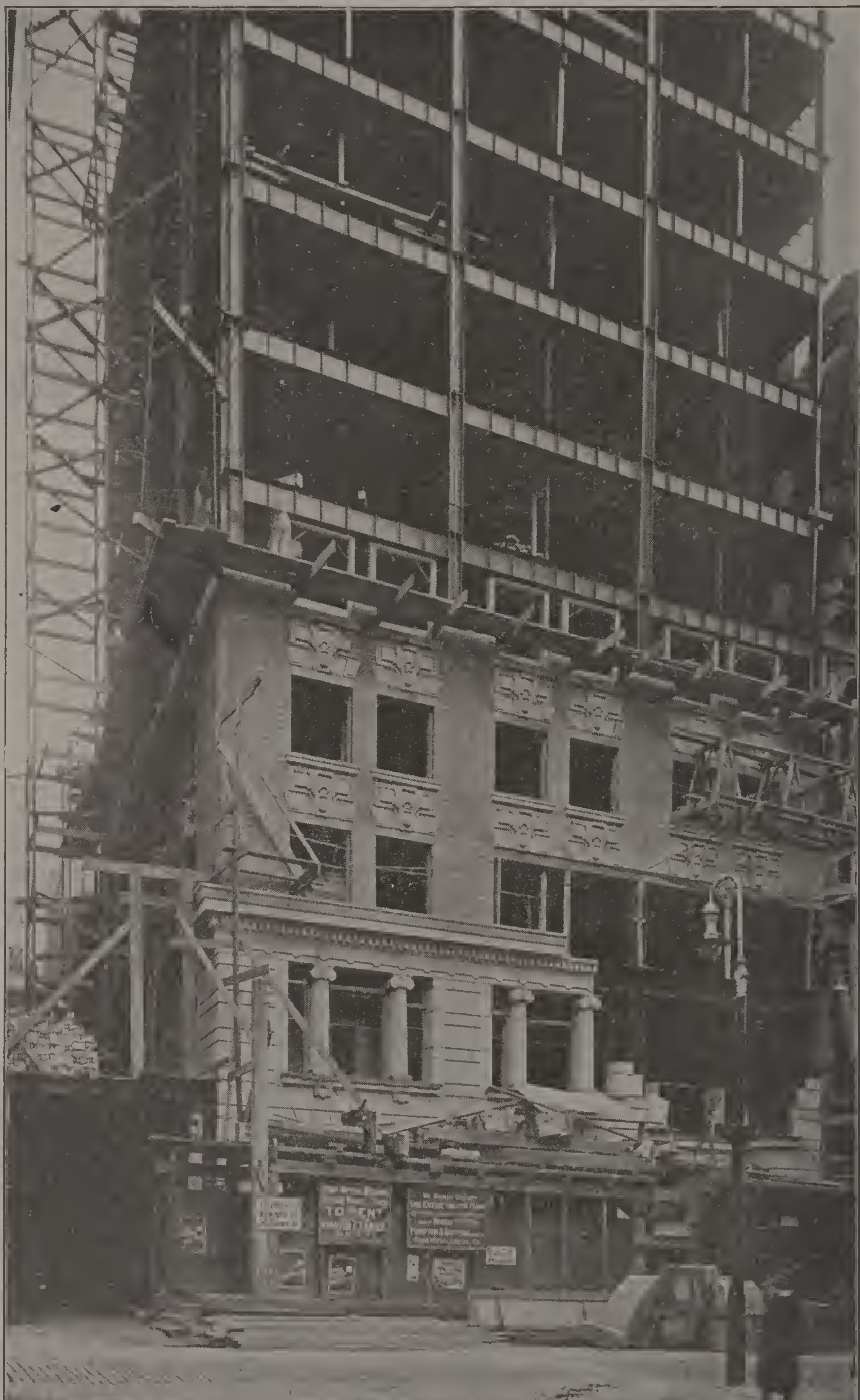


Fig. 164.

Fig. 165

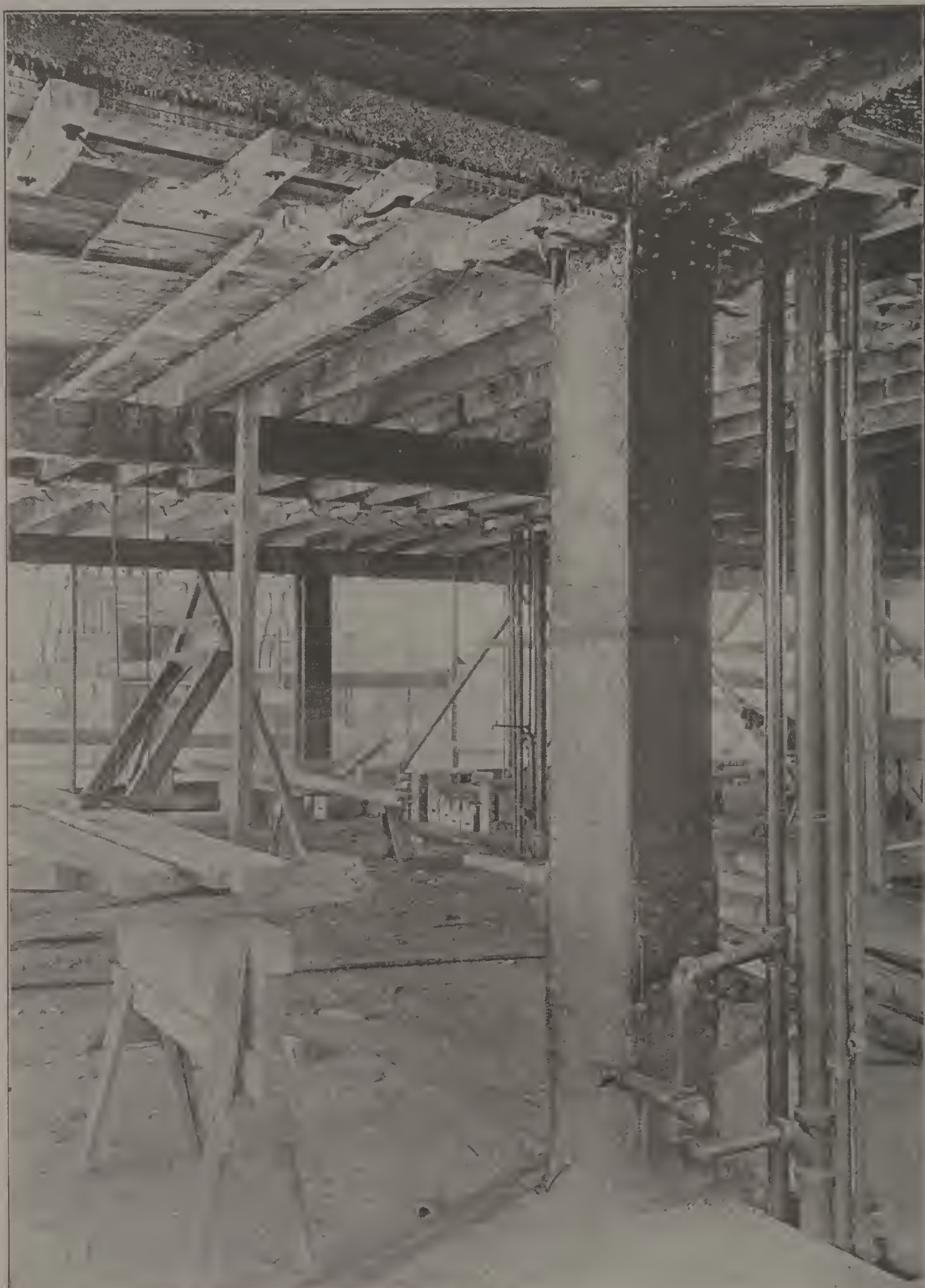


Fig. 166.

Fig. 167.

The cornice brackets and framing are shown in place ready for the cornice when the building shall have reached this stage.

MILL BUILDING CONSTRUCTION.

This term must not be confused with "mill construction." The latter term applies to what is sometimes called "slow burning construction." This is a construction which is the result of the standardizing of requirements and recommendations of the Insurance Underwriters. It applies to a construction in which the walls are of brick, the interior posts of hardwood and of a size generally not less than 8 inches, the floor of heavy wooden girders with hard-wood floor timbers spaced about 5′ — 0″ center to center and 3″ or 4″ of hard-wood floor planks; while this construction is largely of wood the size of the timbers makes them slow burning to a certain degree. Modifications of this construction in varying degrees exist, in which steel replaces some of the wooden members, and from this to the all steel and brick construction. In some cases the spacing of columns and required floor loads make it desirable to use steel or iron columns and steel girders, the floor beams remaining wood, however. In other cases crane loads and other special requirements make steel members more advantageous than the wood. The possibility of reducing the brickwork to a minimum, by carrying all loads on a steel frame, and thus giving large window areas, caused a further development of the steel mill construction. Underwriters object to steel framed mills where the steel is left unprotected and thus exposed to speedy collapse in case of fire. The additional cost of fire-proofing generally results in its omission, however.

Special Features. Mill building, and by this term is included machine shops and all classes of manufacturing buildings, must always be treated according to the requirements and conditions peculiar to the case. Details and capacities cannot be as well standardized as in the case of other classes of buildings, because there are generally features or combinations of features peculiar to the case. For this reason, the required loading should be accurately determined and the details carefully studied. Heavy loads should be brought directly on columns or over girders if possible, rather than supported by shelf or side connections.

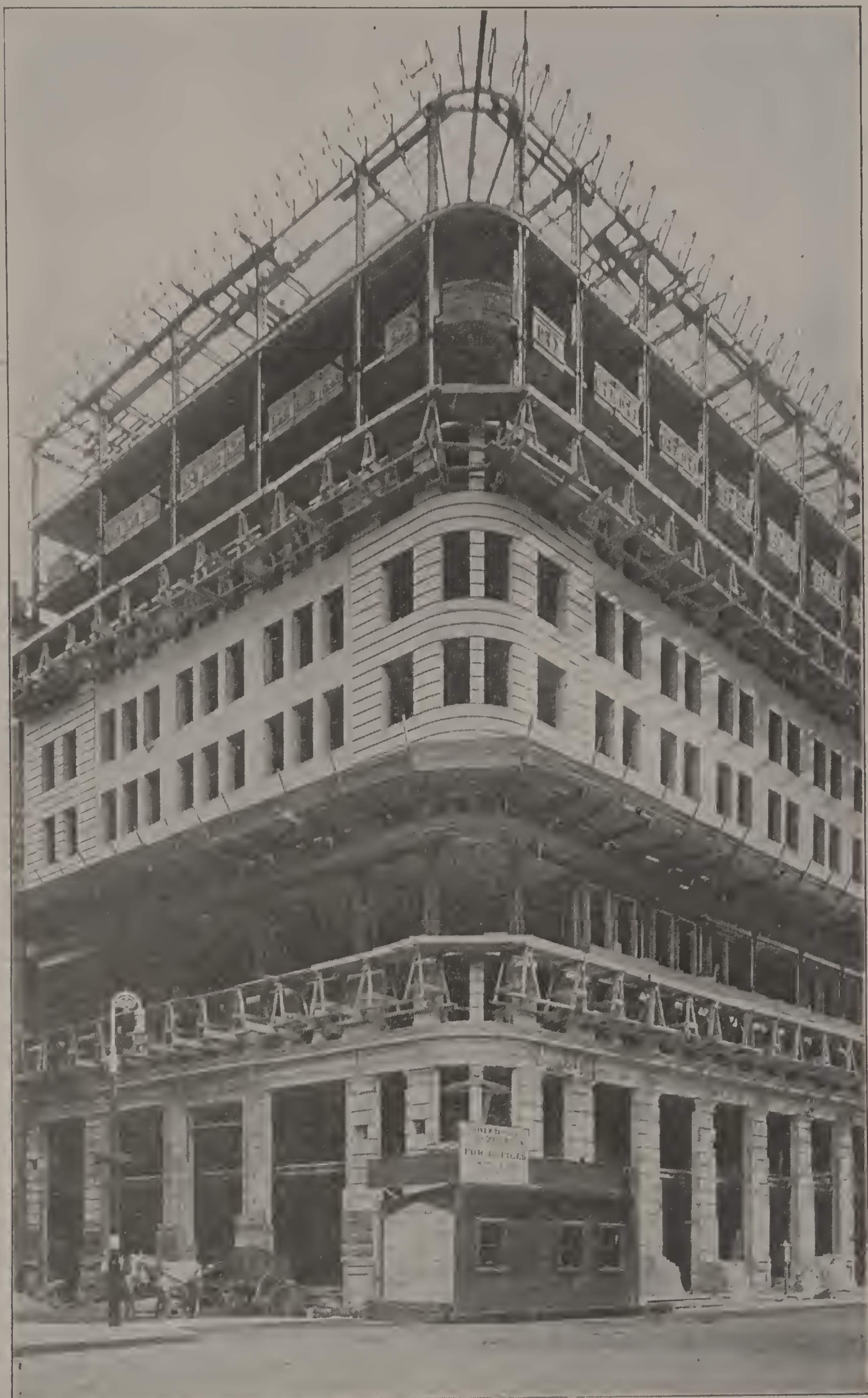


Fig. 168.

Where the building is of the shed construction, that is, with no floors or a very high first story, special provision for strains must be made. Trusses are generally connected rigidly through their whole depth and also by knee braces to the columns. Wind struts at the eaves and at intervals between these and ground are provided. A continuous brace at the ridge, and diagonal bracing in certain bays between the trusses is required.. With certain types of buildings, longitudinal trusses or braces between the main braces are also required. Before details of the different connections met with in this class of construction can be made, the student must become familiar with the general types of construction. While only a few of the more common forms can be given, they will serve as a basis for more complete study of the different types.

Figs. 169 to 174 show general features and details of a building of the shed type.

Fig. 169 shows the side framing, the openings, diagonal bracing, eave strut and columns.

Fig. 170 shows a plan of the columns and trusses, and the bracing between. Fig. 172 shows the end-wall framing, and Fig. 171 is a cross-section showing the type of trusses and the bracing to the columns.

Fig. 173 shows a detail of the walls and the columns. These walls are for protection against weather only, and are not designed to stiffen the steel frame which is sufficiently braced together itself.

Fig. 174 shows the anchorage of the ends of the trusses if solid walls were used in place of the steel wall columns.

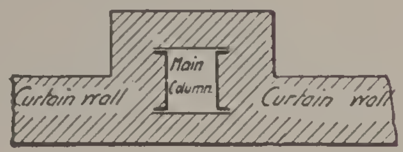
Figs. 175 to 177 show a machine shop steel frame with pin connected trusses. Generally trusses of this character are riveted, but occasionally they are pin connected.

Fig. 175 shows the cross-section with low wings along the side walls and a high central portion to provide room for a travelling crane. This central portion is lighted by a monitor at the top as shown; the windows in the end walls are also indicated.

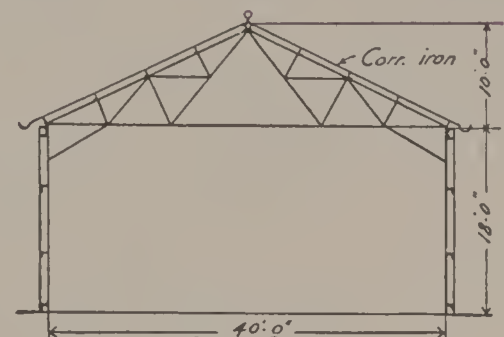
The columns are braced together and to the trusses and the whole frame is self-supporting. The crane runs on a track girder which is supported by a separate column. This is of advantage

Brick Wall for protection against Weather only.
Steel Frame Self Supporting.

MILL BUILDING
DIAGRAM OF FRAMING AND BRACING
Scale $\frac{1}{16}$" = 1'-0"

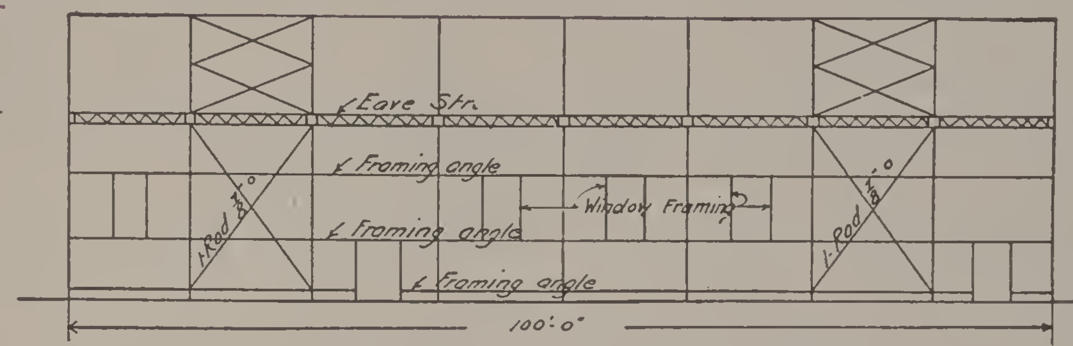


Section through Side Walls
Fig. 173.

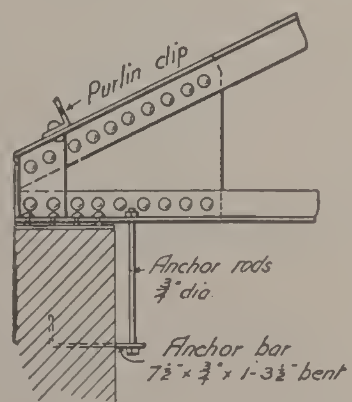


Cross Section

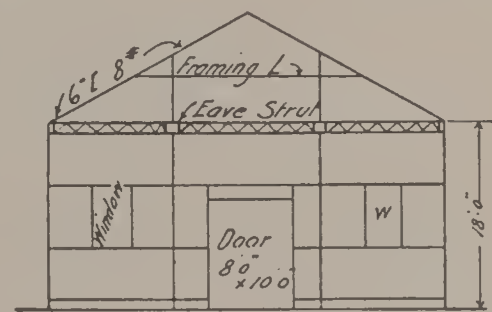
Fig. 171.



Side Elevation of Framing and Bracing
Fig. 169.

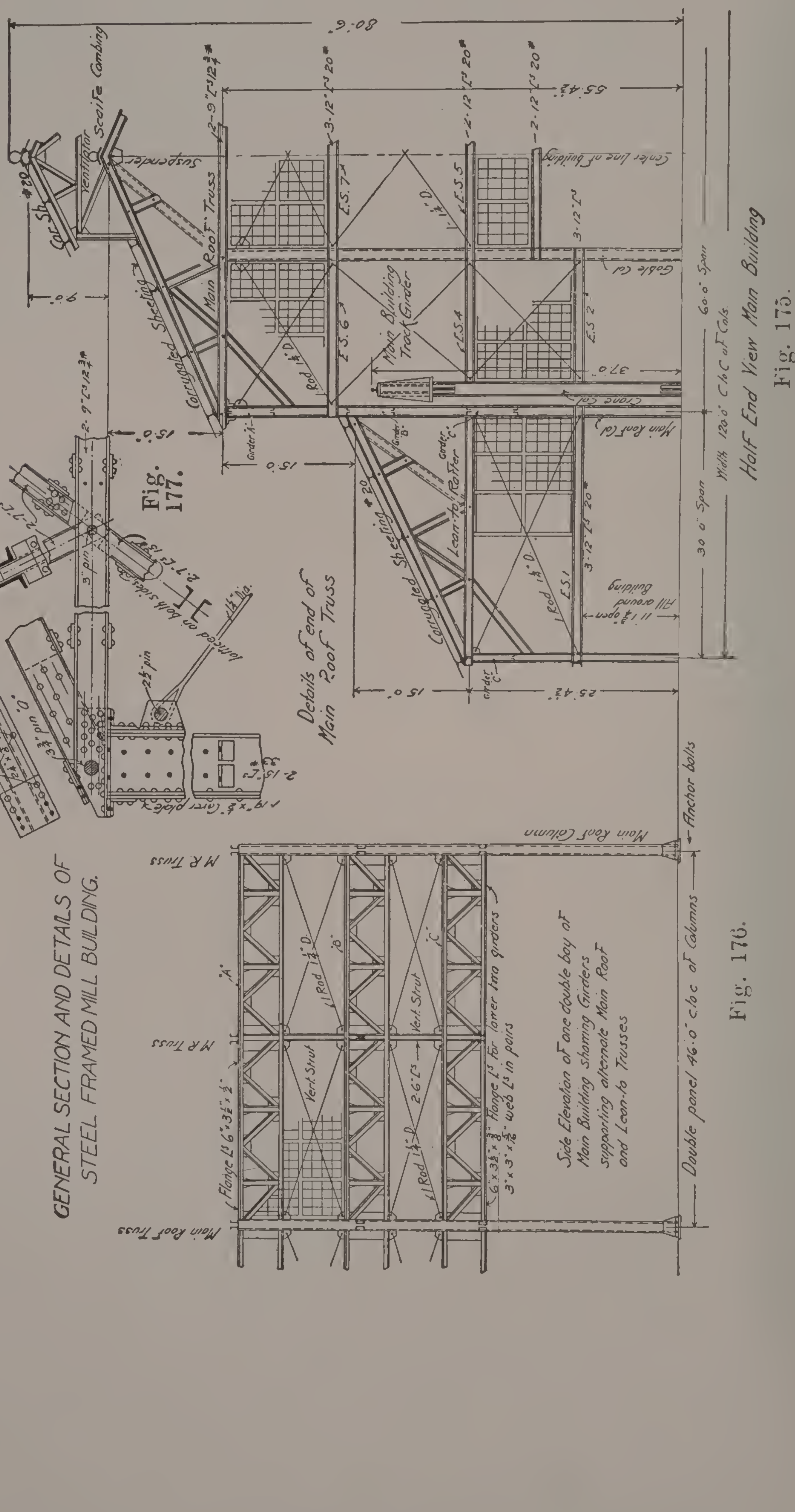


Detail of Wall End of Truss.
Fig. 174.



End Frame
Fig. 172.

Plan showing roof bracing
Bracing shown in solid lines is between rafters
" " " dotted " " " bottom chords
Fig. 170.



GENERAL SECTION AND DETAILS OF STEEL FRAMED MILL BUILDING.

Fig. 175.

Fig. 176.

Fig. 177.

because it allows the column to be placed directly under the load instead of on a bracket which would cause heavy eccentric loading.

Fig. 176 shows a partial elevation of the side. The columns

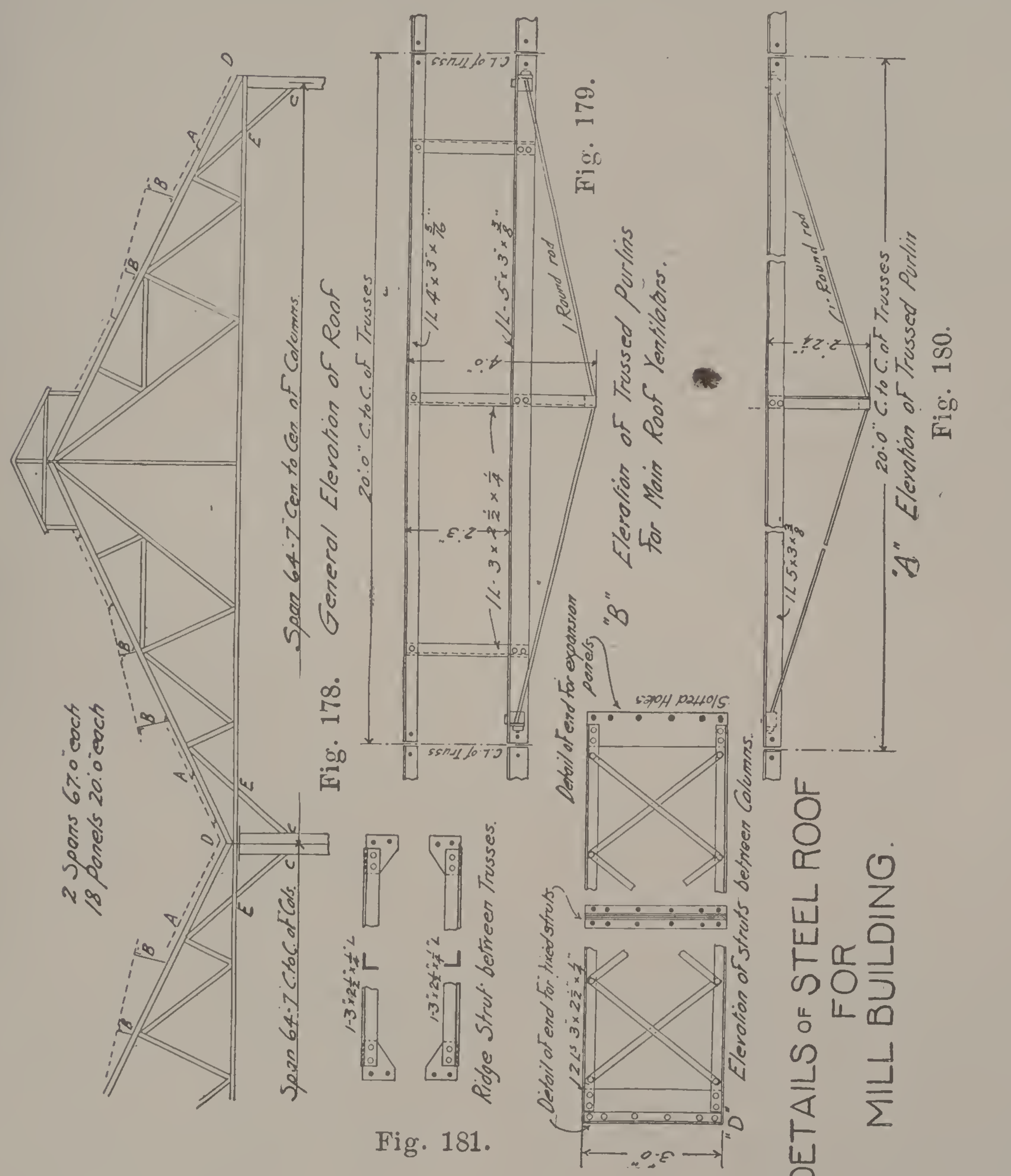


Fig. 178. Fig. 179. Fig. 180. Fig. 181. Fig. 182.

DETAILS OF STEEL ROOF FOR MILL BUILDING.

are placed under every other truss only; the intermediate cross trusses are therefore supported by longitudinal trusses shown by Fig. 176. These trusses serve also to give the necessary lateral stiffness to the frame.

Fig. 177 shows a detail of the ends of these trusses and the

connection to the columns and of the bracing to the columns and trusses.

Figs. 178 to 183 show the outlines and some details of a light mill building having a double pitched roof as shown by the eleva-

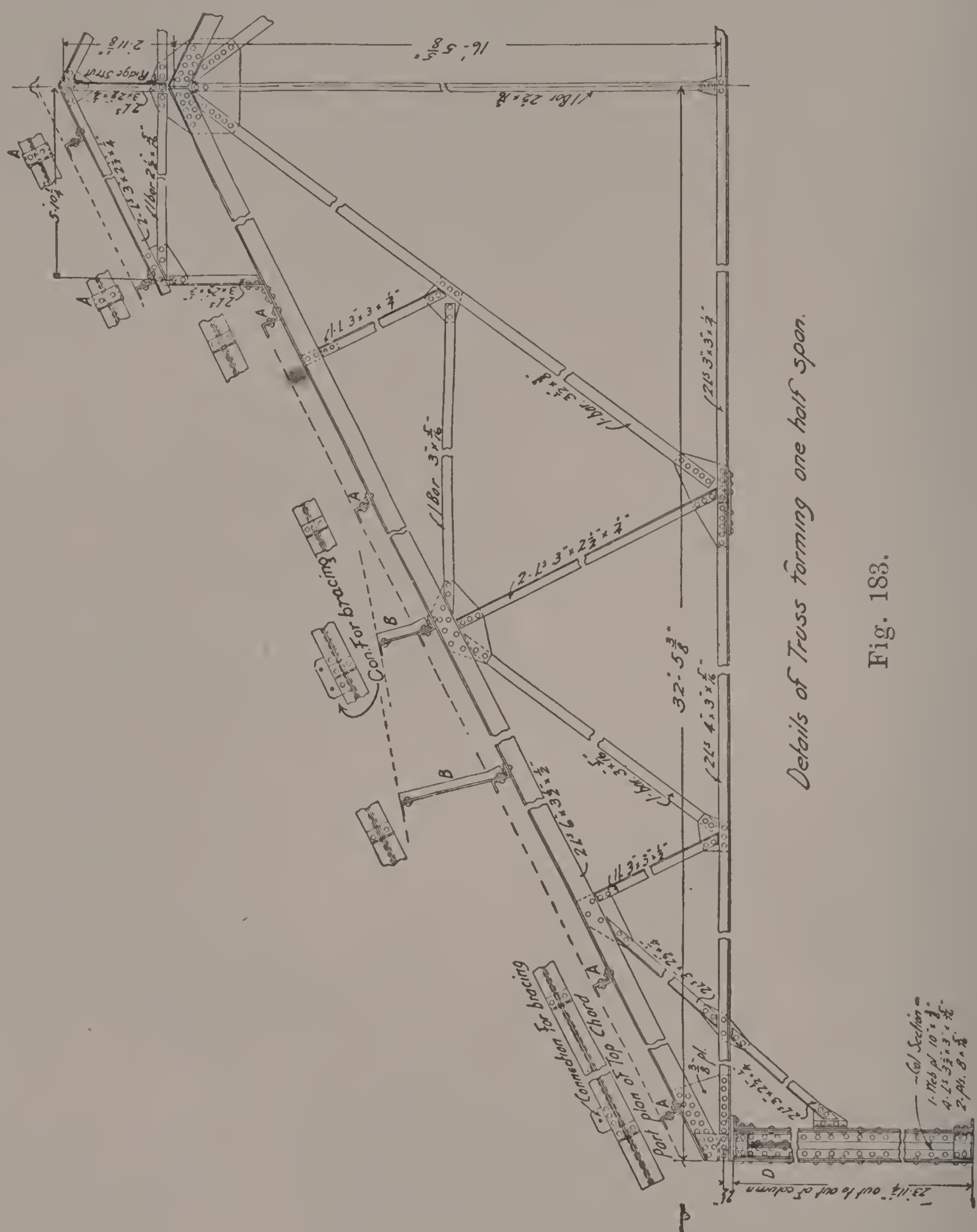
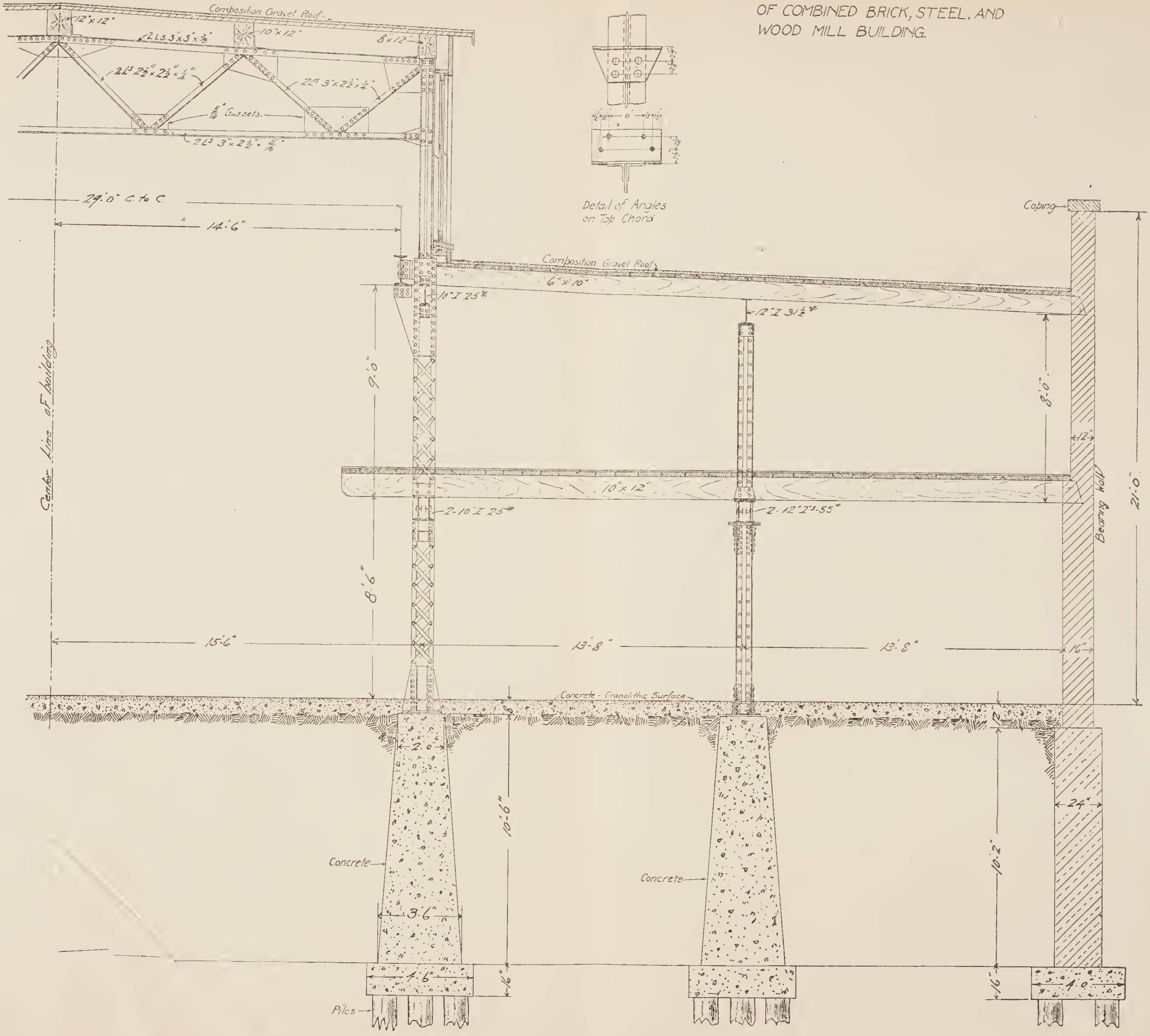


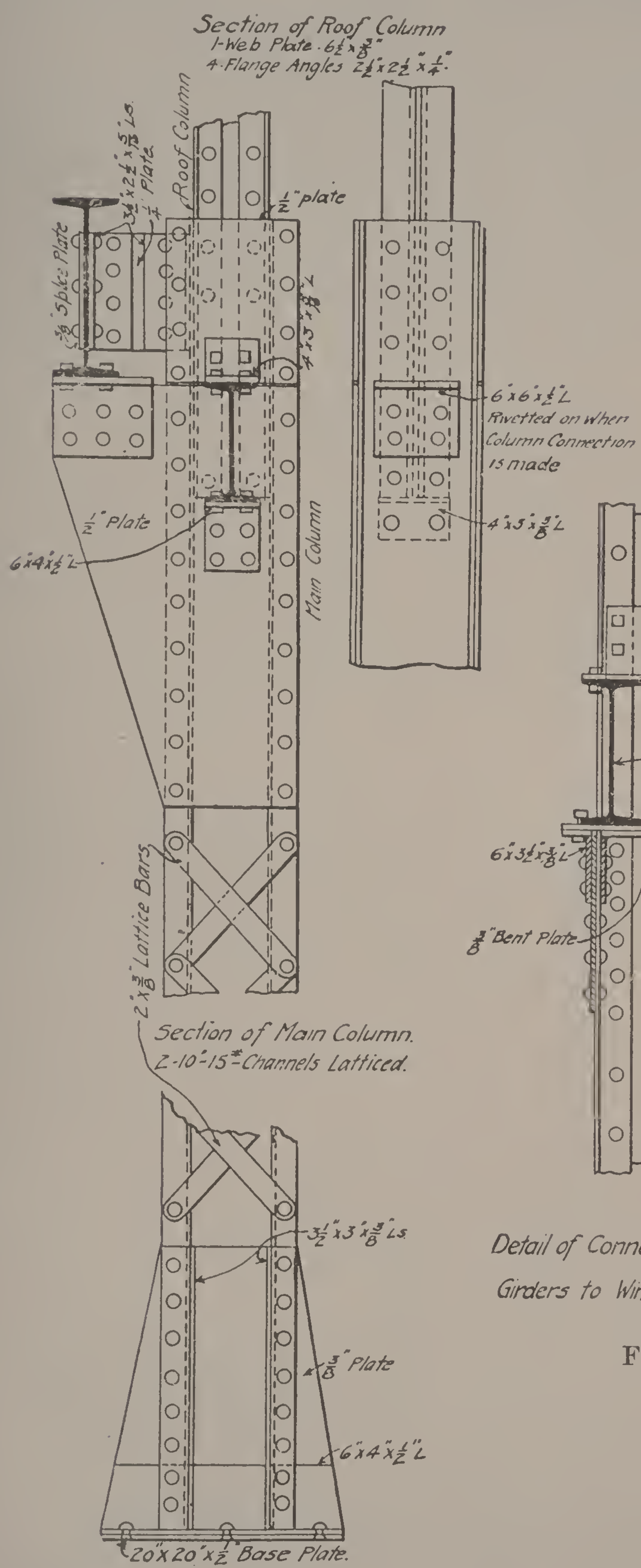
Fig. 183.

tion, Fig. 178. This elevation has letters indicating the positions of the different types of purlins shown by Figs. 179 to 182.

As there are skylights on this roof, purlins "B" have special framing. The regular purlin is "A," and "D" shows the wind

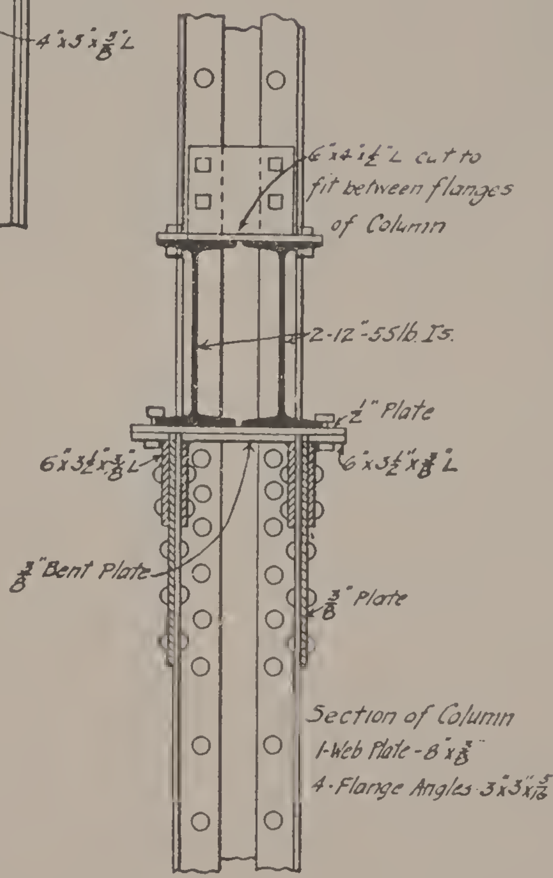
GENERAL SECTION AND DETAILS OF COMBINED BRICK, STEEL, AND WOOD MILL BUILDING.





Detail of Main Column

Fig. 186.



Detail of Connection of Floor Girders to Wing Columns.

Fig. 187.

struts between the columns; there is also a wind strut at the ridge.

Fig. 183 is a detailed elevation of one-half of the main truss, and of the connection of the purlins to the truss.

Figs. 184 to 187 show general features and details of a combined wood and steel frame mill building. This form is used quite extensively. The main columns, trusses and girders are of steel; the roof purlins and floor beams of wood, and the walls of brick.

Fig. 185 shows the detail for securing the wood purlins to the trusses.

Fig. 186 shows the main column which carries a bracket for a light crane. This column, on account of the eccentric crane connection, is made of the two channels latticed as shown; in order to get a stiff connection of roof truss to the upper section of column, and also, because of the light load, a column of four angles and a web was desirable. This upper column, therefore, sets down inside of the channel column and is riveted to it as shown by the details.

Fig. 187 shows the connection of the girders in the wings to the columns; the double beams coming at right angles to the web made it necessary to use deep shear plates across the flanges of the column in order to give support to the bracket and provide for the eccentric strains.

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View showing method of setting steel by hand. Note that the lifted column will rest on top of a column just above the floor line, seen to the right of the place where the lifted column rests on the ground. The bottom of the column rests between cover-plates, to which the column is riveted. The projection shown near top of lifted column is a bracket or shelf to receive a girder.

STEEL CONSTRUCTION.

PART III.

DEFINITIONS AND ABBREVIATIONS.

In all structural steel detailing certain abbreviations are so commonly used that it is essential at the outset for the student to be familiar with them. The more common are given below:

Pl.	=	Plate
⊏	=	Channel.
L	=	Angle.
T	=	Tee.
o	=	Round Rod, and when this mark follows a dimension, as for example, $\frac{3}{4}''$ o, it indicates a $\frac{3}{4}''$ diameter round rod.
□	=	Square.
T. B.	=	Turnbuckle.
O. H.	=	Open Hearth.
R. W.	=	Roadway.
S. W.	=	Sidewalk.
R. & L.	=	Right and Left.
Hex.	=	Hexagon.
H. P.	=	Hard Pine.
Y. P.	=	Yellow Pine.
Blt.	=	Bolt.
U. H.	=	Under Head.
T. & G.	=	Tar & Gravel (also used for tongued and grooved). The right meaning can generally be inferred from the place in which the abbreviation occurs.
Riv.	=	Rivet or Rivets.
Csk.	=	Countersunk.
Cor. I.	=	Corrugated Iron.
Anch.	=	Anchor.
Fill.	=	Filler.

Str.	=	Stringer.
F. B.	=	Floor Beam.
C. I.	=	Cast Iron.
Std.	=	Standard.
Sepr.	=	Separator.
W. G.	=	Wheel Guard.
c. to c.	=	Center to Center.
o. to o.	=	Out to out, or, outside to outside.
Fl.	=	Flange.
Lat.	=	Laterals.
Diam.	=	Diameter.
R.	=	Radius.

The following definitions apply to pieces often met with in detailing and should be fully understood.

Lag Screws. These are used for connecting wooden construction, and their principal use, so far as the structural draftsman is interested, is for fastening guard rails to plank flooring on highway bridges, or to cross ties on railroad bridges, or wood purlins on roof trusses.

Fitting-up Bolts. This term is applied to bolts used to connect parts of a member, or to connect members to each other, prior to riveting. The bolts are removed and rivets driven in their stead. In making out the shop lists where work is to be erected, a number of these bolts must be included, and about 10% more should be ordered than will appear to be necessary, in order to allow for waste. Fitting-up bolts are used in the shop during the assembling of the parts of any member of a structure.

Drift Pins. These are merely tapered steel pins used for aligning the rivet holes so that fitting up bolts may be inserted. Drift pins are also used in many cases to correct inaccuracies in the punching of the several parts of a member. If the holes do not *match*, so that the rivet can be driven through, the drift pin is first driven through and the edges of holes forced out so as to allow the rivet to be inserted. This is a use of drift pins which is not allowed by any first-class specifications, nevertheless it is often done, unless the shop work is rigidly inspected.

Pilot Nuts. A pilot nut is a tapered end which is temporarily screwed on to the end of a pin in order to effect a passage for it

through the pin holes of two or more members which are to be connected in the field. These are, of course, only needed in pin connected structures, but must not be overlooked in making out shop orders and shipping lists, and at least one must be sent for each size of pin used in the structure.

Split Nuts. Owing to lack of room it is sometimes impossible to use a standard nut, and in such cases a thin split nut of about one-half the thickness of a standard nut may be used.

Plate Nuts. For the ends of large pins the nuts are sometimes made from plate cut to hexagon shape and tapped out to fit the threads on the ends of the pins.

Lomas Nuts. These are for use on the ends of large pins such as are used in bridge work. The pins are generally turned down to a smaller diameter at the ends, and these small ends threaded. A Lomas nut grips these threaded ends and projects over the shoulder of the pin. For dimensions and weights of Cambria standard pin nuts see Cambria Handbook, page 336.

Clevis Nuts. On page 334 of Cambria Handbook are shown sketches of clevis nuts, and table giving dimensions, etc., is given. As will be seen in the sketch, the screw ends entering the clevis nut allow the effective length of rod to be adjusted.

Sleeve Nuts. On page 333 of Cambria Handbook is found an illustration and table of dimensions, etc. The purpose of sleeve nuts, as will appear from the illustration, is to allow rods to be adjusted as to their length when the ends are connected to pins or bolts.

Turnbuckles. An illustration of an open turnbuckle is shown on page 332 of Cambria Handbook. Turnbuckles are used the same as sleeve nuts.

Tie Rods. Tie rods are plain rods with screw ends and nuts on each end, and they are used between the beams supporting fireproof floors to tie the beams together and to hold them in position while the fireproofing is being put in place. The tie rods also stiffen the I-beams laterally. The sizes of rods used for this purpose are usually $\frac{5}{8}$-in. diameter to 1-in. diameter. See Fig. 207.

Loop Eye Rods. Rods which are connected to other parts of a structure by pins are provided with loops made by bending the rod around to conform to a circle of same diameter as the pin, and welding the end into the body of the rod. The distance from the

center of pin to the junction of the end of loop with the main rod is usually made about two and a half times the diameter of the pin which loop is to connect over. See Fig. 188.

Forked Eye Rods. Sometimes it is desirable to have a rod connecting to a pin fastened so as to bring an equal strain on each side of another rod or part connecting to the same pin. In such cases it is necessary to make a forked eye instead of a single loop. See Fig. 188.

Upset Rods. When rods are threaded at the ends, the cutting of the threads diminishes the effective area of the rod and consequently weakens it. To maintain the same strength throughout, the rod is "upset" at the ends before the ends are threaded, and the amount of extra thickness so provided allows the threads to be cut, and leaves after cutting a net area equal to that in the body of the rod.

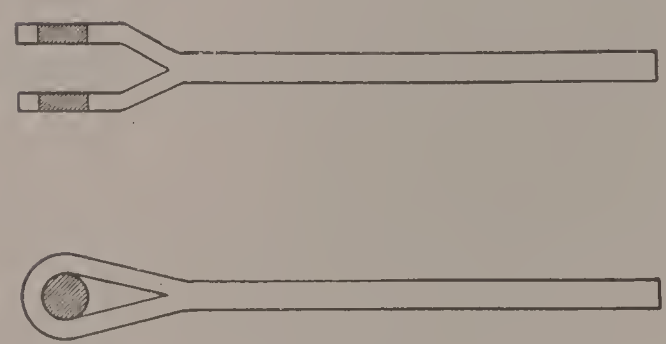


Fig. 188.

Upsetting is done by a machine which takes hold of the heated end of the rod when at a cherry-red heat and compresses the metal for the required length into a cylindrical end larger in diameter than the main body of the original rod. See pages 326 to 329 of Cambria Handbook.

Plain Rod. The expression "plain rod" is simply the negative of the term "upset rod", which has just been refined, or, in other words, a "plain rod" is *not* upset.

Standard Threads. Rods and bolts are generally provided with standard threads the dimensions of which will be found on page 316 of Cambria Handbook.

Right-hand Threads. When the threads of a bolt or rod are cut so that if, when looking at the end of the bolt or rod and turning the nut from left to right, the nut moves from you, or is screwed on the threads, then such threads are referred to as right-hand threads.

If the threads are cut so that the reverse is true then they are "left-hand threads".

Eye Bars. These are used in pin connected trusses and structures to take care of tensile strains. The heads at each end are

formed by upsetting machines and the pin holes afterward bored out. See page 331 of Cambria Handbook for dimensions, etc., of standard eye-bar heads.

Batten Plates. In Fig. 225 a batten plate is placed at each end of the strut on the top and bottom of the flanges. It is used merely to tie together the two parts of the strut. Batten plates (also called tie plates) are used generally wherever lacing is used in order to tie the parts of a member together at each end of the lacing. The Pencoyd Iron Works specifications for railroad bridges gives the following in regard to tie plates:

"All segments of compression members, connected by latticing only, shall have tie plates placed as near the ends as practicable. They shall have a length of not less than the greatest depth or width of the member, and a thickness not less than one-fiftieth of the distance between the rivets connecting them to the compression members".

Chas. Evan Fowler, in his Specifications for Roofs and Iron Buildings, refers to tie plates as follows:

"Laced compression members shall be stayed at the ends by batten plates having a length equal to the depth of the members".

The rules given in various specifications are somewhat different as regards the length and thickness, being determined by each authority merely on his own judgment of what will prove satisfactory. There is no method of proportioning batten plates except in accordance with such specifications as may be furnished in relation to the particular job of work in hand.

Lacing. Single lacing is used on the girder shown in Fig. 225, but if two systems of lace bars are used crossing each other and riveted at their intersections, it is called double lacing This is only used on very heavy members. Single lacing is usually placed at an angle of about 60 degrees with the axis of the member, while double lacing is placed at about 45 degrees to the axis.

The size of lace bars to use is somewhat a matter of judgment, but certain rules have been established by common practice and experience which it is well to observe when practicable. Chas. Evan Fowler's specifications give the following:

The sizes of lacing bars shall not be less than that given in the following table. When the distance between gauge lines is

6 in. or less than	8 in.	$1\frac{1}{4}$ in. $\times$ $\frac{1}{4}$ in.
8 in. " " "	10 in.	$1\frac{1}{2}$ in. $\times$ $\frac{1}{4}$ in.
10 in. " " "	12 in.	$1\frac{3}{4}$ in. $\times$ $\frac{5}{16}$ in.
12 in. " " "	16 in.	2 in. $\times$ $\frac{3}{8}$ in.
16 in. " " "	20 in.	$2\frac{1}{4}$ in. $\times$ $\frac{7}{16}$ in.
20 in. " " "	24 in.	$2\frac{1}{2}$ in. $\times$ $\frac{1}{2}$ in.
24 in. or above, use angles.		

They shall generally be inclined at 45 degrees to the axis of the member, but shall not be spaced so as to reduce the strength of the member as a whole. Where laced members are subjected to bending, the size of the lacing bars shall be calculated, or a solid web plate used.

Shop Drawings. In making shop drawings, the outlines of the member (in other words, the "picture" of it) should be done in fairly heavy lines, so as to show up clearly on the blue prints, and the dimension lines should be very light so that they will not be confused with the outlines of the members. All distances should be given from center to center, wherever possible. Dimensions from the edge of an angle, beam, or plate, should never be given unless there is a special reason for so doing; because all rolled shapes vary in the width of the flanges, and Z-bars also vary in height. The reason for this variation is that different sizes are rolled by the same set of rolls and the difference is made in the spacing of the rolls. See Figs. 25, 26, 27 of Part I. Also, angles of a thickness of one-half inch or more vary somewhat in the length of legs unless they are given what is called a finishing pass or rolling which is not always done.

Make all drawings on the dull side of tracing cloth with a No. H H or a No. H H H pencil. After the drawing is completed the pencil marks are easily removed with a piece of sponge rubber.

Do not draw out your work on paper first and then trace it. You will find that this is a waste of valuable time. Learn to draw directly on the tracing cloth, as you will be expected to do when you begin work in an office. You will need the following outfit in the way of drafting instruments and equipment:

1 T-square, at least 20 in. long.
2 Triangles, 1 of 45°, the other 60°.

1 Small drawing board, about 18 by 24 in.
$\frac{1}{2}$ dozen small thumb tacks.
1 Ruling pen.
1 Circular pen or spring bow pen.
Tracing cloth.
1 Triangular boxwood Architect's scale 12 in. long.
1 Bottle of Higgins' American drawing ink.
$1\frac{1}{2}$ dozen Gillot's pens No. 303.
1 No. H H pencil.
1 No. H H H pencil.
1 Copy of Cambria Handbook, Edition 1904.

PURPOSE AND USE OF DETAILS.

A shop drawing is a drawing which gives all the information necessary to lay out, cut, punch, and rivet the piece shown. It is the medium by which instructions are conveyed from the engineer's office to the shop. It must convey full, accurate and explicit instructions for every operation. It must be so clear and explicit that no further explanations are needed to enable the shop to correctly interpret it, and the information must be given in such form that only one interpretation is possible. The draftsman making a shop drawing must constantly bear in mind that the man at the shop will work entirely from this drawing; that he does not have access to the sources of information which are consulted by the draftsman in making the drawing, and that what might be clear in connection with these other drawings will be blind or uncertain to the shop man not familiar with them. The draftsman should further understand that it is distinctly the duty of the shop man not to read into the drawing anything not there, and that consequently the responsibility is entirely upon the draftsman to make his drawing so complete that such action will be unnecessary and impossible. Neatness in execution of a shop drawing is desirable, but accuracy and clearness are absolutely essential.

Shop drawings differ from general detail drawings in that they do not show the different parts of construction assembled, but cover only one piece. For instance, an engineer making a drawing to send to the drafting room where the shop details are to be prepared, would show a column with the girders and beams framing into it,

just as they would appear when assembled. In this way he would establish the relations of the different members and would determine the character of the connections and any special features of the details. The draftsman detailing for the shop, however, would make the column on one sheet, each beam and girder on separate sheets, and the different members forming the whole structure would appear only as individual pieces, their relations one to the other being given by an assembly or erection drawing.

In a large shop the columns, beams, and girders would be fabricated in entirely distinct departments and the men in the different departments would not know that those different pieces when assembled, fitted into each other. The responsibility for correctly laying out these pieces so that they will fit together is upon the draftsman.

Measurements on shop drawings are always carried out as close as one-sixteenth of an inch, and sometimes to one thirty-second. An error of one-sixteenth may be sufficient to make it impossible to assemble the pieces in erection, as steel cannot be cut and drilled at the building except at considerable expense of time and money. Such errors are costly.

The student should clearly understand the importance of the work of the shop draftsman and should always be imbued with the idea that he is the last authority to pass upon all the various points determining the instructions of the shop and the last sentinel to discover and prevent errors. Drawings are almost always checked by some other than the man who makes them, but no man will make a successful draftsman unless he does his work without a thought of being saved from errors by the checker.

The making of templets, and the way in which a shop uses a detail drawing have already been explained. The draftsman should always detail as far as possible in accordance with standard shop practice, as in this way much templet work can be eliminated and thus time and expense saved, and the work will be more quickly fabricated because of the familiarity of the shop with the details. The standard forms differ somewhat in the different shops, but the Carnegie standards are essentially the same as all others; these have been given in Steel Construction, Part I. A great many conditions arise in which standard forms cannot be used, in which cases as simple details as practicable should be employed.

Scales Used in Details. Details of plate and box girders and of trusses are almost invariably made to scale, generally $\frac{3}{4}$, 1 or $1\frac{1}{2}$ in. to the foot. Details of columns are generally made to scale as far as the connections for beams and the head and foot of columns are concerned. The length along the shaft from top to bottom and between connections at different levels is generally not to scale.

Details of beams are rarely drawn to scale, but the position of holes and of shelf angles, etc., are shown in the proper relation to each other and to the whole beam. That is, if the beam shown is a 12-in. beam 16 ft. long, the elevation of the beam might be drawn to a scale of $1\frac{1}{2}$ in. to the foot as regards the height of beam, while as regards the length it might be drawn at no definite scale, simply made to come within the limits of the sheet. In locating holes in this elevation, if there was a horizontal line of holes in the center of the beam it should show in the center of space limiting the height of beam; if another line 2 in. off from the center, it should be shown at $\frac{1}{6}$ of the depth from the center line. Similarly to spacing holes along the length of the beam a set of holes centrally located as regards the length should show in the center of the sketch, and another set 2 ft. from the center should show $\frac{1}{8}$ of the whole length from the center.

In other words, the beam is detailed according to the scale of the sketch which represents the beam, but this will not be the same scale vertically as horizontally and will not be the same scale for any two sketches.

The reason for the above absence of scale in beam sketches is that these details are almost invariably made on a standard size of sheet, say 12 × 18 in. One sheet may have beams varying in depth from a 7-in beam to a 15-in. beam, and in length from 6 ft. to 20 ft. To accommodate all such varied conditions to the same size sheet it is necessary to adopt a standard size of sketch representing all sizes and lengths of beams, and locate details on this sketch simply by the eye, so as to show the details in proper relations as outlined above. In many drafting offices these beam sheets are printed with a blank elevation and plan and end view of a beam ready for the draftsman to fill in the details.

In the case of columns, girders and trusses, this practice would not do, as the details are too complicated and it is necessary to show all details exactly in their true relation in order to make them clear.

In the case of columns this can be done on a standard size sheet, generally 12 × 30 in. or 18 × 30 in. Girder sheets and truss sheets generally vary in size with the particular conditions of each case.

The first operation necessary is to draw out the outlines of the member to be detailed, showing a side elevation and plan, or end view and sections where necessary to clearly show all the work to be done. Make no *unnecessary* drawing; as, for instance, if a side elevation and plan will clearly express all the work to be done, do not spend any time making an end view or sections. If, on the other hand, an elevation and a cross section will enable you to show everything, then do not make any plan, as, in general, it is less work to make a cross section than a plan.

The above should be followed with caution, as it is necessary to be very sure that all the views required to give a clear understanding of the details are given.

Rivet Holes, Etc. Holes for rivets are either simply punched, or punched to a smaller size than that actually required and reamed out to the full size, or else the holes are drilled. Rivet holes are seldom drilled, except under special specifications, owing to the increased expense. On almost all work at present the holes are simply punched. In case reaming or drilling is required the shop drawing must indicate it clearly.

Where the holes are simply punched the usual specification is that the diameter of the punch shall not exceed the diameter of the rivet, nor the diameter of the die exceed the diameter of the punch by more than one-sixteenth of an inch.

Where the holes are punched and afterward reamed, the usual specification is "All rivet holes in medium steel shall be punched with a punch $\frac{1}{8}$ in. (sometimes $\frac{3}{16}$ in.) less in diameter than the diameter of the rivet to be used, and reamed to a diameter $\frac{1}{16}$ in. greater, or they may be drilled out entire".

The effective diameter of the driven rivet shall be assumed the same as before driving, and in making deductions for rivet holes in tension members, the hole will be assumed one eighth of an inch larger than the driven rivet.

The pitch of rivets is generally specified about as follows: "The pitch of rivets shall not exceed sixteen times the thickness of the plate in the line of strain, nor forty times the thickness at right angles

to the line of strain. The rivet pitch shall never be less than three diameters of the rivet. At the ends of compression members it shall not exceed four diameters of the rivet for a length equal to the width of the members."

Rivets and Riveting. Rivets are spoken of as "shop rivets" or "field rivets" according to whether they are to be driven in the shop or in the field during the erection of the work. It is sometimes impossible to drive rivets by machine in the shop, owing to their location being inaccessible for the riveter. In such cases they must be driven by hand and are referred to as hand-driven rivets. Driving rivets by hand is necessarily more expensive than if done by machinery, and it is part of the duties of a competent structural draftsman to so design the details as to require the least possible driving of rivets by hand, whether in the shop or field. In erecting large jobs the field riveting is often done by machine riveters. There are numerous types of machine riveters, the principal power used being either compressed air or hydraulic power.

In order that rivets may be driven by the riveting machine it is necessary to have a certain amount of clearance from the heads of other rivets which project from the other leg of an angle if the two rivets are opposite or nearly opposite each other. This is shown in Fig. 189, together with a table giving sizes of rivet heads and clearances for machine driving. At the bottom of this table please note that a must not be less than $\frac{1}{4}$ in. + $\frac{1}{2}\,h$. Suppose we wish to drive two rivets, each $\frac{7}{8}$ in. diameter, and both to have full heads exactly in the same line in the two legs of an angle. Now, if we desire to know how close we can drive the rivet in the horizontal leg to the back of the angle, we first find the value of h for a $\frac{7}{8}$ in. rivet, which is $1\frac{7}{16}$ in. Then $a = \frac{1}{4}$ in. + $\frac{1}{2}$ ($1\frac{7}{16}$ in.) = $\frac{31}{32}$ in. Add this to the height of the rivet, which, for a $\frac{7}{8}$ in. rivet is $\frac{21}{32}$ in., and we have $1\frac{5}{8}$ in. as the distance from the center of the rivet in the horizontal leg of the angle to the side of the vertical leg of angle nearest to this rivet. But all measurements to locate the position of rivets are given from the *backs* of angles; hence we must add the thickness of the angle in order to find where the rivet in the horizontal leg should be spaced. Suppose the angle to be $\frac{3}{8}$ in. thick, then $1\frac{5}{8}$ in. + $\frac{3}{8}$ in. = 2 in. would be the least distance from the back of the angle that we could drive either rivet in order to have the riveting machine clear the other.

Rivets could, however, be spaced nearer to the back of the angle if the rivets are "staggered", *i.e.*, if those in the vertical leg were spaced so as to come in between the two adjacent ones in the horizontal leg. An example of staggered rivets is shown in Fig. 233.

Conventional Signs. In erecting some classes of structural steel work, especially in light highway bridges and small roof truss jobs, the connections are often made with bolts instead of rivets. The rivets used for structural steel work are round headed (sometimes called "button head") rivets. It is necessary sometimes to flatten the heads of rivets after the rivet is driven, and before it has

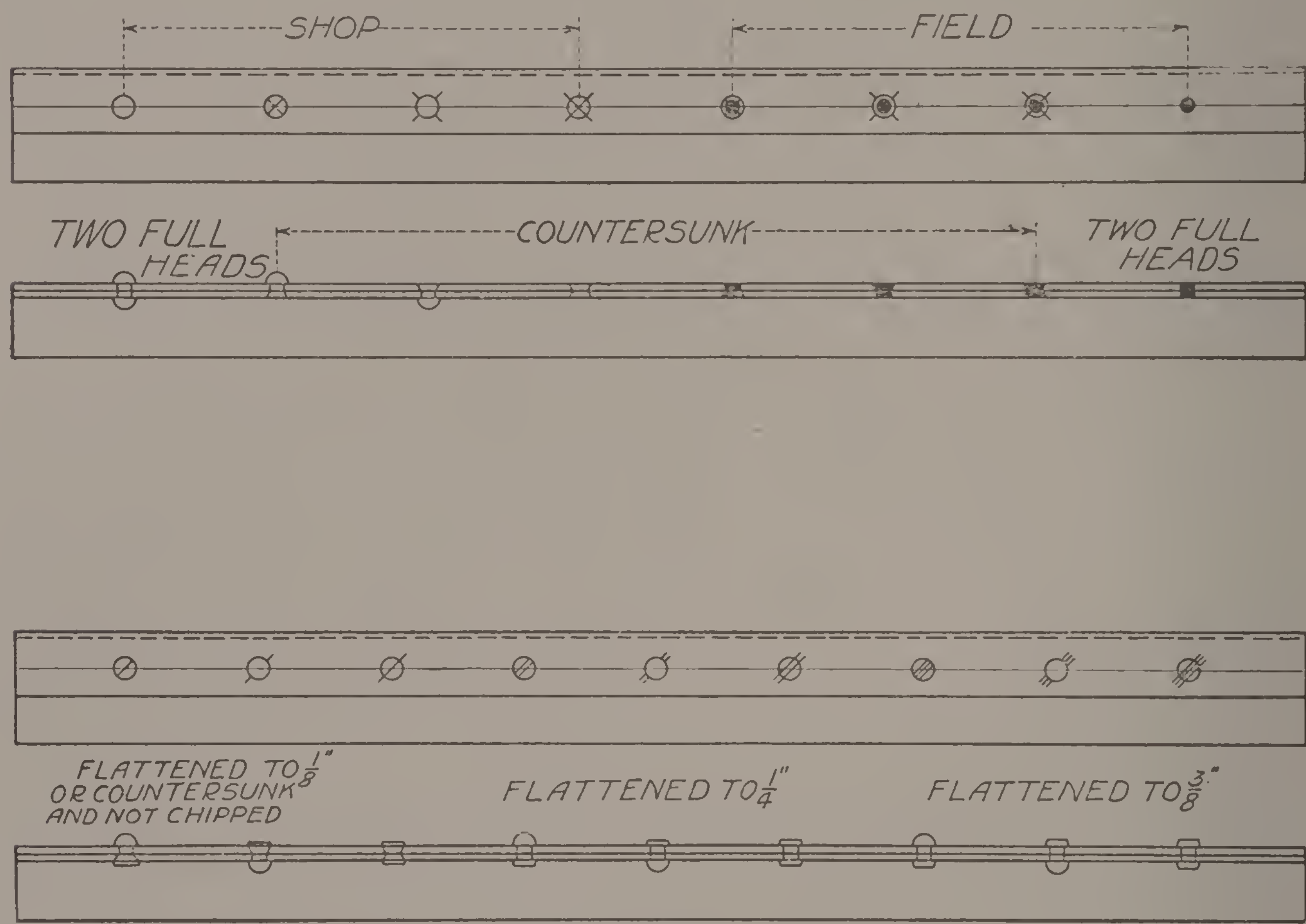


Fig. 189.

had time to cool. This is done by simply striking the red hot head of the rivet and flattening it to the extent desired. Wherever a flattened head would interfere with some connecting part of a structure it is necessary to countersink the heads, sometimes on one end of the rivet and sometimes on both ends. Fig. 189 shows conventional signs for representing the different kinds of rivet heads desired, and this code is in general use in the United States.

It is very important to show on all shop drawings the diameter of rivets to be used in the work, and if different sizes of rivets or rivet

holes for field rivets occur in the same member, then these must be indicated on the drawing by a note prominently displayed so that the shop men may readily find it and avoid error. The sizes of rivets generally used for structural steel and bridge work are $\frac{5}{8}$ in., $\frac{3}{4}$ in., or $\frac{7}{8}$ in. in diameter, although special work may require smaller sizes, and occasionally rivets 1 in. in diameter are used for very heavy work.

Rivets are made with one head formed, and the shank of the rivet must be long enough to project through the parts to be joined, and far enough out on the other side to form a full perfect head when subjected to the pressure of the machine. After the rivet has been heated to a cherry red it is inserted in the rivet hole and the riveter is placed so that the cap fits over the head already formed, and the other jaw of the machine presses against the protruding shank of the rivet and forms the head. It is desirable that riveting machines be made to hold on to the two ends of the rivet with the full pressure until the rivet partially cools.

The terms "rivet pitch" and "rivet spacing" refer to the distances center to center between rivets. For example, if the rivets are spaced 3 in. apart for a certain distance along a member of a structure, we refer to the rivets for this portion of the member as being of three-inch pitch. Fig. 190 gives the lengths of rivets required for a given "grip".

PROBLEMS.

1. Given an 18-in., 55-lb. I-beam with a $4 \times 4 \times \frac{1}{2}$-in. shelf angle riveted on one side; what length of $\frac{3}{4}$-in. rivet should be ordered for riveting this angle on in the field?

2. In Fig. 187 of Part II, is shown a 12-in. beam girder bolted to a cap angle on a column; what length of bolts should be ordered for this connection?

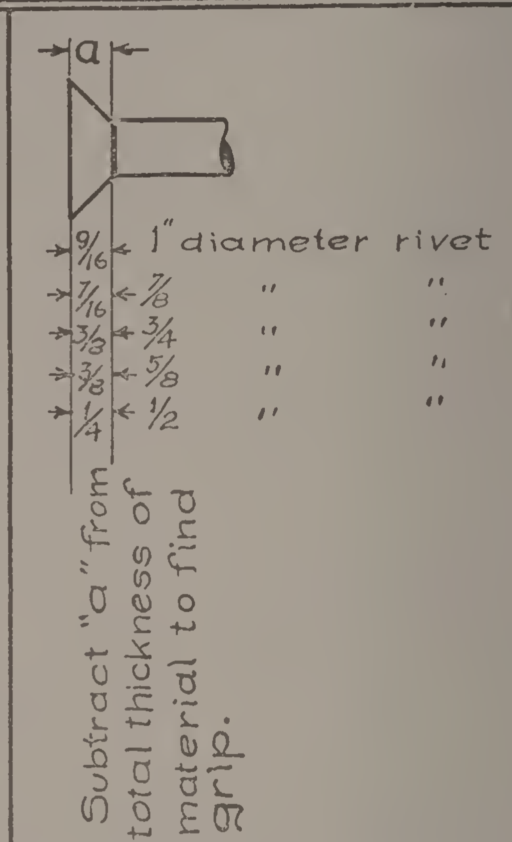
3. If the beams shown in Fig. 187 are $6\frac{1}{2}$ in. center to center, and are bolted up, using standard cast iron separators, what lengths should be ordered for these separator bolts?

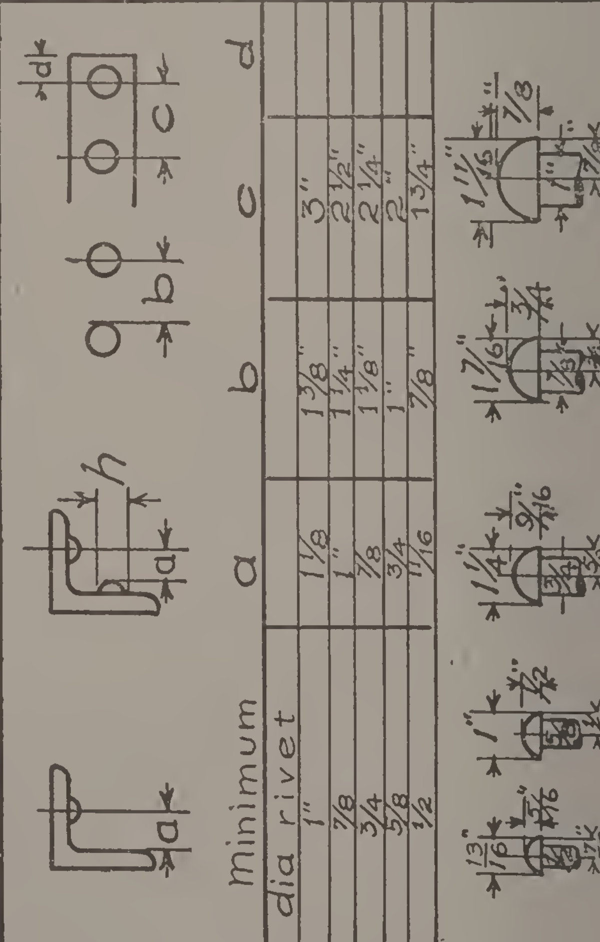
4. Suppose a 12-in., 40-lb. beam and a 7-in., 15-lb. beam are framed opposite each other on a 15-in., 60-lb. girder; if standard connection angles are used, what length of $\frac{7}{8}$-in. field rivets should be ordered for the connection of the beams to the girder?

Length of field rivets for holes punched or drilled $\frac{1}{16}''$ larger than diameter of rivets

dia. rivet	7/8	3/4	5/8	1/2
Grip.	length	length	length	length
3/8	1 7/8	1 3/4	1 5/8	1 1/2
1/2	2	1 7/8	1 3/4	1 5/8
5/8	2 1/8	2	1 7/8	1 3/4
3/4	2 1/4	2 1/8	2	1 7/8
1/8	2 3/8	2 1/4	2 1/8	2
1	2 1/2	2 3/8	2 1/4	2 1/8
1 1/8	2 5/8	2 1/2	2 3/8	2 1/4
1 1/4	2 3/4	2 5/8	2 1/2	2 3/8
1 3/8	2 7/8	2 3/4	2 5/8	2 1/2
1 1/2	3 1/8	3	2 7/8	2 5/8
1 5/8	3 1/4	3 1/8	3	2 7/8
1 3/4	3 3/8	3 1/4	3 1/8	3
1 7/8	3 1/2	3 3/8	3 1/4	3 1/8
2	3 5/8	3 1/2	3 3/8	3 1/4
2 1/8	3 3/4	3 5/8	3 1/2	3 3/8
2 1/4	3 7/8	3 3/4	3 5/8	3 1/2
2 3/8	-4-	3 7/8	3 3/4	3 5/8
2 1/2	4 1/8	4	3 7/8	3 3/4
2 5/8	4 1/4	4 1/8	4	3 1/2
2 3/4	4 3/8	4 1/4	4 1/8	4
2 7/8	4 1/2	4 3/8	4 1/4	4 1/8
3	4 3/4	4 5/8	4 1/2	4 1/4
3 1/8	4 7/8	4 3/4	4 5/8	4 1/2
3 1/4	5	4 7/8	4 3/4	4 5/8
3 3/8	5 1/8	5	4 7/8	4 3/4
3 1/2	5 1/4	5 1/8	5	4 7/8
3 5/8	5 3/8	5 1/4	5 1/8	5
3 3/4	5 1/2	5 3/8	5 1/4	5 1/8
3 7/8	5 5/8	5 1/2	5 3/8	5 1/4
4	5 3/4	5 5/8	5 1/2	5 3/8
4 1/8	5 7/8	5 3/4	5 5/8	5 1/2
4 1/4	6	5 7/8	5 3/4	5 5/8
4 3/8	6 1/4	6	5 7/8	5 3/4
4 1/2	6 1/2	6 1/4	6	
4 5/8	6 5/8	6 3/8		
4 3/4	6 3/4	6 1/2		
4 7/8	6 7/8	6 5/8		
5	7	6 3/4		

"a" must not be less than 1/4 in. + 1/2 h.





Minimum dia rivet	a	b	c	d
1"	1 1/8	1 3/8"	3"	
7/8	1"	1 1/4"	2 1/2"	
3/4	7/8	1 1/8"	2 1/4"	
5/8	3/4	1"	2"	
1/2	11/16	7/8"	1 3/4"	

Fig. 190.

5. If it is necessary to drive two rivets of $\frac{5}{8}$ in. diameter exactly opposite in the two legs of an angle $3 \times 3\frac{1}{2} \times \frac{7}{16}$ in.; how close to the back of the angle can the rivets be spaced?

Strength of Joints. The student should now become familiar with the method of calculating the strength of joints and connections. We will take first the connection of one beam framed to another. The rivets in the connection, of course, are the only means of transmitting the load from the beam to the girder. There are two sets of these rivets, one set through angles on the end of the beam to be carried and the other set through the outstanding legs of these angles and through the web of the girder. The load must go from the beam through the first set of rivets into the connection angles, and then from the angles through the second set of rivets into the girder.

The rivets through the angles securing them to the web of the beam are subject to failure in two ways. (1) The rivet might break along the two planes coincident with the faces of the web of the beam, thus allowing the beam to drop between the two angles—this method of failure is called "*shearing*" of the rivets. (2) The rivets might crush the metal of the web of the beam on the upper semi-circumference of the rivets; this is called failure by "*bearing*."

In designing a connection, the number of rivets is determined by whichever provision against these two methods of failure gives the greatest required number. The strength of a rivet as regards shearing and bearing is called its *value*, and in order to determine the number of rivets to carry a given load in connections of this character, it is only necessary to determine the value to be used for one rivet. This value is determined in the following way:

DETERMINATION OF SHEARING VALUE OF RIVETS.

The resistance of a rivet to shearing along one plane is the area of the rivet multiplied by the shearing strength of the metal per unit of area.

If d = the diameter in inches of the rivet

S = the ultimate shearing strength in pounds per sq. in.

then V = the ultimate shearing value in pounds.

$= .7854\, d^2\, S.$

For the working value of the rivet a certain proportion of S is used and this varies with the factor of safety required. The safe

value of the shearing strength per square inch of power-driven rivets which is generally used for buildings, is 9,000 pounds, which gives a factor of safety of about six. With rivets three-quarters of an inch in diameter, which is the usual size in building work, the safe shearing value is therefore

$$.7854 \times \tfrac{3}{4} \times \tfrac{3}{4} \times 9000 = 3976 \text{ pounds.}$$

For rivets driven by hand as is done in many cases in assembling the parts in the erection of a building, the safe shearing strength per square inch is reduced to 7,500 pounds. One of the connections illustrated in Fig. 191 is a case of *double shear* for the rivets through the angles and the web of the beam, as there are two planes along which shearing must occur, since the load is distributed by the web of the beam equally between the two angles. The above value of 3,976 must be multiplied by two to give the total resistance of each of these rivets against shearing.

The rivets, however, which go through the outstanding leg of these angles, and through the web of the girder which carries this beam are only in *single shear*, as here there is only one plane between the angles which transmit the load and the web which receives it. The value for these rivets would therefore be 3,976 lb. if power driven, and 3,313 lb. if hand driven.

DETERMINATION OF BEARING VALUE OF RIVETS.

In this case it is the metal which bears on the rivet or which the rivet bears on, which has to be considered; this is in compression and liable to failure, therefore, just as is the metal in a column or the compression side of a girder. The amount of stress which this metal will stand is determined by the ultimate compressive strength per square inch, and the area under compression, which area is the product of the diameter of the rivet and the thickness of the metal or in this case, the web of the girder.

If therefore t = thickness of metal
d = diameter of rivet
C = ultimate compressive strength in pounds per square inch,
then V_b = ultimate bearing value in pounds
$= Cdt$

The safe value usually used for power-driven rivets in building work is 18,000 pounds per square inch; for three-quarter-inch rivets, therefore, the bearing value becomes for a $\frac{5}{16}$-in. web $18,000 \times \frac{3}{4} \times \frac{5}{16} = 4,219$ pounds, and for hand-driven, rivets, 3,516 pounds.

The web of the beam in Fig. 191 is a case of *bearing enclosed*, that is, it is enclosed on both sides by other members, and therefore is stiffened against buckling under compression. The web of the girder is not enclosed, as it is free to buckle on one side. Most authorities allow a slightly greater bearing value, generally about 10 per cent for bearing on metal enclosed.

In designing such a connection as is illustrated in Fig. 191, the number of rivets through the web of the beam would be determined by the bearing value of one rivet, unless the thickness of this web was $\frac{5}{8}$ in. or over, since for all thicknesses less than this the bearing value would be less than the double shear. The number of rivets

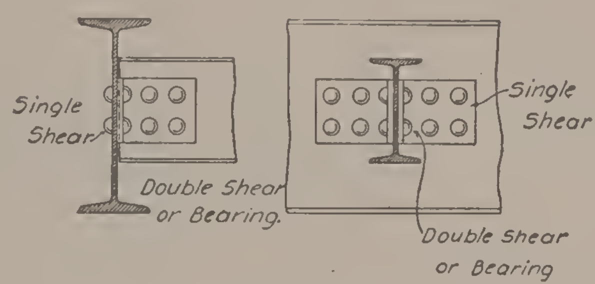


Fig. 191.

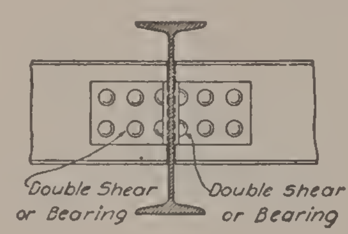


Fig. 192.

through the web of the girder would be determined by the shearing value of one rivet for all thicknesses of webs of $\frac{5}{16}$ in. and over, since for these thicknesses the bearing value is greater than single shear. Where two beams frame into a girder on opposite sides so that the rivets through the girder are common to both beams as shown in Fig. 192, these rivets are in bearing on the web of the girder for the combined load brought by both beams, in double shear for the combined loads, and in single shear for the load from each beam. If these loads were the same for each beam, single shear from the load from one beam would, of course, be equivalent to double shear for the load from both beams; if, however, the loads were greatly dissimilar the greatest load with the single shear value must be used. To illustrate this, suppose we have a 10-in. beam framed on one side of a 10-in. beam and an 8-in. beam framed opposite to it. Suppose the load brought by the 10-in. beam to the girder is 14,000 pounds,

and that by the 8-in. beam 6,000 pounds. Now the web of a 10-in. 25-lb. beam is .31 inches thick, and the bearing value would therefore be $.31 \times 15{,}000 \times .75 = 3{,}487$ pounds, and for the total load this would require six rivets. To carry the load of 14,000 pounds in single shear at a value of 3,313 would require but five rivets, so that the bearing value and the total load from both beams would determine the number of rivets.

If, however, these beams were carried by a 12-in., 40-lb. beam whose web is .46 inches thick, the bearing value would then be 5,175 pounds and this would require but four rivets; in this case the number would be determined by the greatest load and the single shear value of a rivet. Fig. 193 shows a single angle connection which would be determined by the rivet in single shear. It should be noticed that in designing connections a few rivets in excess of the actual number calculated should be used for connections; in general, 20 per cent should be added.

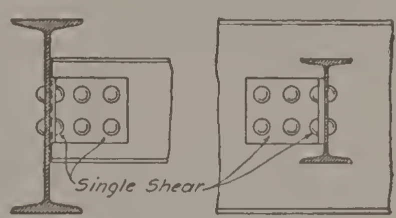


Fig. 193.

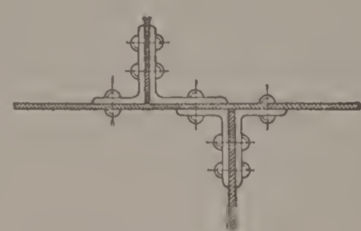


Fig. 194.

PROBLEMS.

1. Suppose that certain rivets to be provided in a connection are in double shear. The rivets are all $\frac{7}{8}$ in. in diameter. The outside plates are each $\frac{1}{2}$ in. thick. What will be the thickness of the inside plate to make the rivet value equal to double shear?

2. Suppose a 6-in., 12.25-lb. I-beam that is 5 ft. long carries a load of 15,000 lb., uniformly distributed. How many rivets $\frac{3}{4}$ in. in diameter, will be required for its connection to the beams at each end, allowing 6,000 lb. per square inch for shear on the rivets, and 12,000 pounds per square inch for bearing?

3. In the preceding problem, how many rivets $\frac{3}{4}$ in. in diameter will be required to attach the connection angles to the 6 in. I-beam? In order to determine this, it will be necessary to first find the thickness of the web of the 6-in., 12.25-lb. I-beam. This can be found by referring to the tables on pages 30 and 31 of Part I. As the

thickness of the webs is there given in *decimals* of an inch, these must be converted into the next smaller common fractions.

4. Given a 12-in., $31\frac{1}{2}$-lb. I-beam 14 ft. long and a 10-in., 25-lb. I-beam 12 ft. long framed opposite to each other to a 15-in., 42-lb. I-beam. If these beams are each loaded to the safe capacity with a uniformly distributed load, what will be the number of $\frac{3}{4}$-in. rivets required for the field connection to the girder, using 7,500 pounds for shear and 15,000 pounds for bearing?

5. In the above problem what will be the number of $\frac{3}{4}$-in. shop rivets required on the end of each beam using 9,000 pounds for shear and 18,000 pounds for bearing?

6. Using the same values and loads as in problem 4, what will be the number of rivets required in each beam, if they do not frame opposite each other?

7. Give the lengths of field rivets and shop rivets required for each connection in each of the cases covered by problems 2, 3, 4, 5, 6.

STANDARD CONNECTIONS.

As previously stated, beam connections to girders and columns are generally made after standard forms for the different size beams. From an inspection of these standard connections it will be seen that 3, 4, 5, and 6-in. beams and channels all have the same number of rivets; 7, 8, 9, and 10-in. sections have the same number; and of the larger beams the different weight beams of a given size have the same number, whether the lightest or heaviest section is used. It is evident that these beams which are of different capacities, would not require the same number of rivets, if the number was calculated for the exact load of each case. It would not be economical, either from the standpoint of time or money, to detail in this way, however, and therefore these standard forms are always used unless peculiar conditions made it impossible to frame with these size angles, or unless because of peculiar conditions of loading, these connections would not be sufficiently strong.

These standard connections are proportioned for uniformly distributed loads with spans commonly used for the different size beams. When beams are used on short spans and loaded to their full capacity, it would be necessary to design special connections with the required number of rivets; the same is true where a concentrated

load comes on a beam very near to one connection. The tables on pages 42 and 43 of the Cambria Handbook give the minimum spans of the different size beams and channels for which these standard connections can be used when the beams are loaded uniformly to their full capacity, based on 10,000 lb. per sq. in. for shear and 20,000 lb. for bearing. For cases of concentrated loading near the ends, no general rule can be given. For all cases of loading on spans shorter than those given by the table, the draftsman should calculate the load on the connections and determine the number of rivets required.

Connection angles are always riveted to beams centrally as regards the depth of web unless conditions make it necessary to raise or lower them. Such conditions arise when certain beams of different depths frame opposite to each other to the same girder. There are standard conditions concerning many of these cases and these are shown in Figs. 135 to 139, Part II. Such connections should be made by changing the position of the angles rather than the spacing of the holes in the angles if possible, so that the standard framings can be used.

Where beams frame on opposite sides of the same girder, but the center lines of the two beams do not lie on the same straight line special size angles and rivet spacing is required. If the distance between the center lines of the beams is less than $8\frac{1}{2}$ in., as shown in Fig. 194 the one line of rivet holes must be common to both beams. The minimum distance between rivets of beams framed to the same side of a girder for which standard connection angles can be used is shown in Fig. 137. In cases where beams are spaced closer than this, a single angle with the required number of rivets is used in the outside of each web; or where there is sufficient depth of girder a shelf angle below the beams can be used. In this case stiffeners fitted to the outstanding leg of the shelf angle should be used, as under deflection the beam will bear near the outer edge of angle and without the stiffeners would tend to break off this leg. The full number of rivets required to carry the load should be put in the stiffeners and shelf, even if angles on the web of the beam are used to hold it laterally. It is not good design to rely on the combined action of two sets of connections, such as a shelf connection described above, and a web connection, to carry a load. In such a case the deflection of the

beam would bring the bearing on the shelf, and this connection would take the whole load; or if the shelf was not stiffened to resist bending under the load, this would throw the load on the web connections. Wherever a shelf with stiffeners is used it should contain enough rivets for the full load.

Where beams frame to deep girders or to columns, even if the connection is made by angles on the web of the beams, it is customary to put a shelf angle under the beam. The student should not confuse this construction with the one just described. The object of such a shelf is to facilitate erection and not to support the beam after the web connection is made. Where such an angle is used, therefore, no stiffeners should be used under the beam, as these would prevent the web connection from performing the work for which it was designed. The draftsman must see that the connection angles are not placed so as to interfere with the fillet of the beam or of the girder. This consideration arises where the connection is raised or lowered on the beam, or where the beam does not frame flush with the girder, or where a small beam frames flush with a large one, as for instance a 5-in. beam to a 24-in. beam. Fig. 36, Part I, gives rules for determining the distance from outside of the flange to the commencement of the fillet. These distances are given also in the Cambria Handbook. It is possible to encroach a little on the fillet but generally not more than $\frac{1}{8}$ in.

The standard form of connections of beams to columns is by a shelf angle with the stiffeners under it, with the required number of rivets, and with a cap angle over the top. The beam is riveted both to the cap and the shelf angles. Generally there are four rivets in each flange—sometimes only two in each flange are used. The shelf angle is usually a $6 \times 6 \times \frac{1}{2}$-in. angle and the cap angle a $6 \times 6 \times \frac{7}{16}$-in. angle where four rivets in the flange are used; if only two rivets are used the outstanding leg would be 4 inches instead of 6 inches. The size of stiffener angles varies with the size necessary to conform to the rivet pitch of the column, and to keep the outstanding leg of the stiffener the required distance from the finished line of the column. As stated previously, the deflection of the beam tends to throw the load near the outer edge of the angle and therefore the stiffener should come as near this edge as is practicable. Another point to be considered in choosing the size of stiffeners is to bring the out-

standing leg as near as practicable under the center of the beam as this is the portion of the shelf loaded by the beam. It is not always practicable to do this, however, and sometimes two stiffeners are used coming a short distance each side of the center of the beam.

A good many designers use only one stiffener under a beam or girder, and as the load to the stiffener comes from the outstanding leg, this brings a moment on the rivets through the other leg of the stiffener. For usual sizes of beams, there is probably ample strength in the rivets to provide for this moment. It is better design, however, to use two stiffeners back to back, with rivets connecting the outstanding legs, as shown in Fig. 217. This avoids the strain due to the moment on the rivets and also distributes the load to the column symmetrically with regard to the axis, instead of entirely on one side. These points are of very great importance where heavy girders or unusually heavy concentrated loads are concerned. Special column connections will be taken up later on.

The connections of beams to double beam girders, involve the consideration of a number of practical points peculiar to each case. These beams are generally bolted together with only a slight space between the flanges, and if the girder rests on a column, the holes must be arranged where they are accessible. In general this would be in the outside flanges unless the end of the girder was exposed so that the inside flanges could be reached.

Where beams frame to such a girder they cannot be riveted unless it is possible to rivet all the lines of such connections to each beam comprising the girder before they are brought together and bolted up. Where there were several lines of such girders it would be difficult to do this for all of them. In many cases, therefore, these connections have to be arranged for bolts to go through both beams of the girder. Where double beam girders frame into another girder the connection can only be made by single-angles on the outside of the webs, unless the beams are spread far enough apart to allow bolts or rivets on the inside to be reached. If the girder carrying the double beams is deep enough a shelf connection can of course be used, and this would be preferable to the single-angle connection.

Connections by angles on only one side of the web, as shown in Fig. 193, should always be avoided if possible, as they are subject to a bending moment on the rivets in the same way noted for single

stiffeners. Where such a connection must be made sufficient extra rivets should be used to provide for this moment. The remarks in regard to double beam girders apply also to girders made up of three and four beams. In these cases, however, there must be room for connection angles on the inner beams, and if the connection cannot be made when the beams are bolted together, it must be arranged so that these beams can be erected before the outside ones. In such an arrangement it is obvious that the standard form of cast iron plate separators could not be used very readily unless rods were used through the separators instead of bolts.

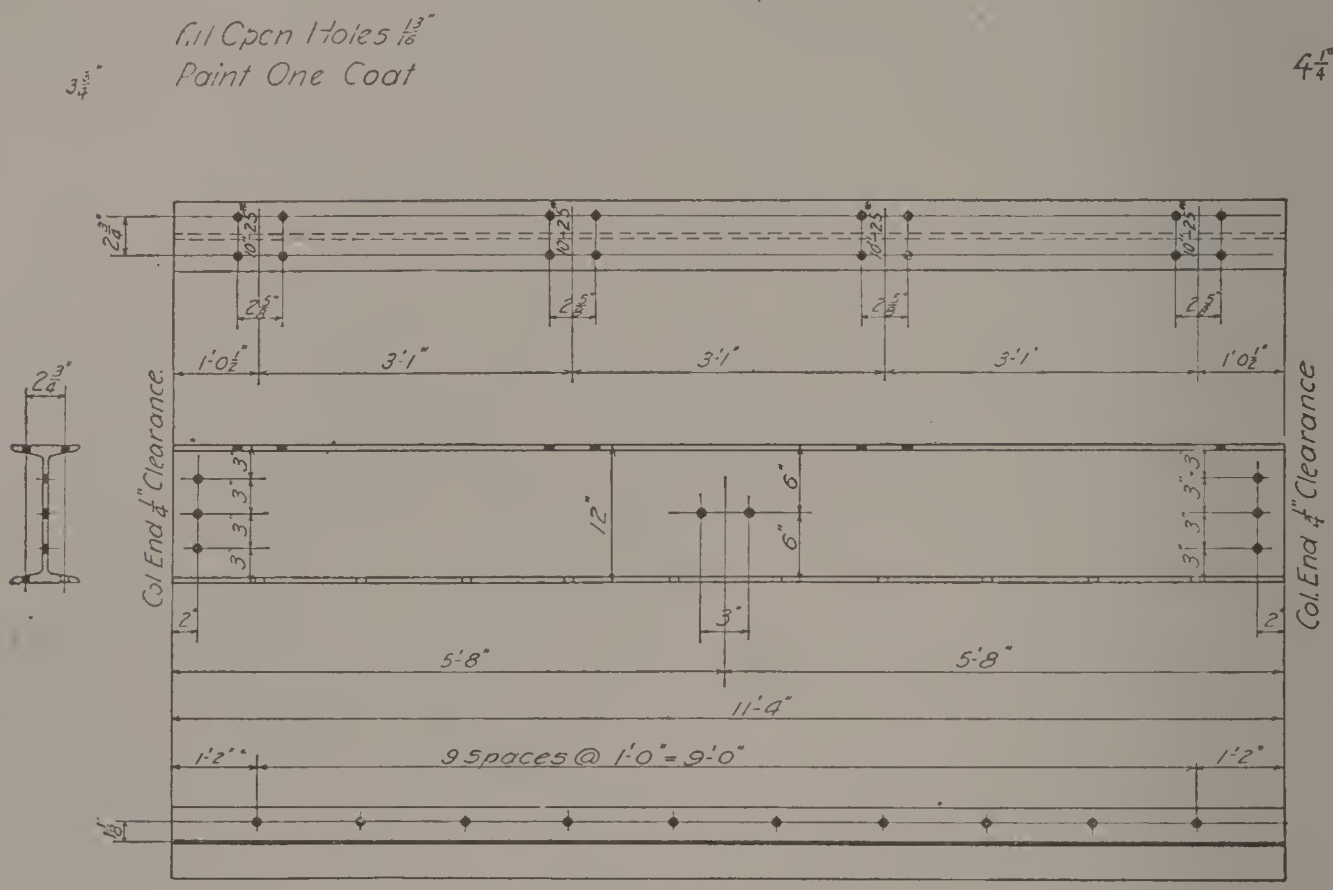
In Figs. 131 to 140, Part II, are shown cases of special framing to which the student should refer again and become thoroughly familiar with.

Where different sizes of beams frame opposite to the same girder it is necessary to change the position of the framing angles on the beams in order to use standard connections in each case. These changes in position are generally made to conform to standard practice, which is illustrated in Part II and which in general is as follows: In all cases except where one of the beams is a 7-in. beam, the first hole is $3\frac{1}{4}$ in. from the flanges which are flush with each other, and standard angles are used. Where one of the beams is a 7-in. beam and the other is either a 6, 8, 9, 10 or 12-in. beam the first hole is $2\frac{1}{2}$ in. from the flush flanges; for a 12-in. beam the first hole is $2\frac{3}{4}$ in.

Fig. 190 shows the Carnegie code of conventional signs for rivets. It is important to follow the code in use by the particular shop for which the drawings are intended, as only by the use of such signs can elaborate notes be avoided.

Illustrations of Details. Fig. 195 shows a detail of a punched beam. Note that there should always be a single overall measurement on the sketch. Groups of holes, as for instance holes for connections of other beams, as shown in the top flange and the web, are located by fixing the center of the group. The reason for this is that the beam on which is the framing connecting to the holes is located by its center, and therefore it is important to locate this exactly. If the holes are symmetrical with regard to the center it is not necessary to dimension each hole from the center, but simply to give the distance between them, corresponding with the distance in the outstanding legs of the connection angles on the beam framing to this one.

In the case of a channel it is the back of web rather than the center which is always located. For the holes for connections of a channel, therefore, the position of the back of channel is fixed, and then each hole in the group forming the connection must be located with respect to the back of channel as the group is not central with regard to the back. For an example of this see Fig. 196. It always adds to the clearness to put near each group of holes forming a connection for other beams the size of beam or channel connecting to it. Holes at ends of beams for connecting to columns or for anchors



4-12"-31.5# Beams 11'-4" long o.a. Mark "2nd. Floor" Nos. 51, 74, 96, 108.

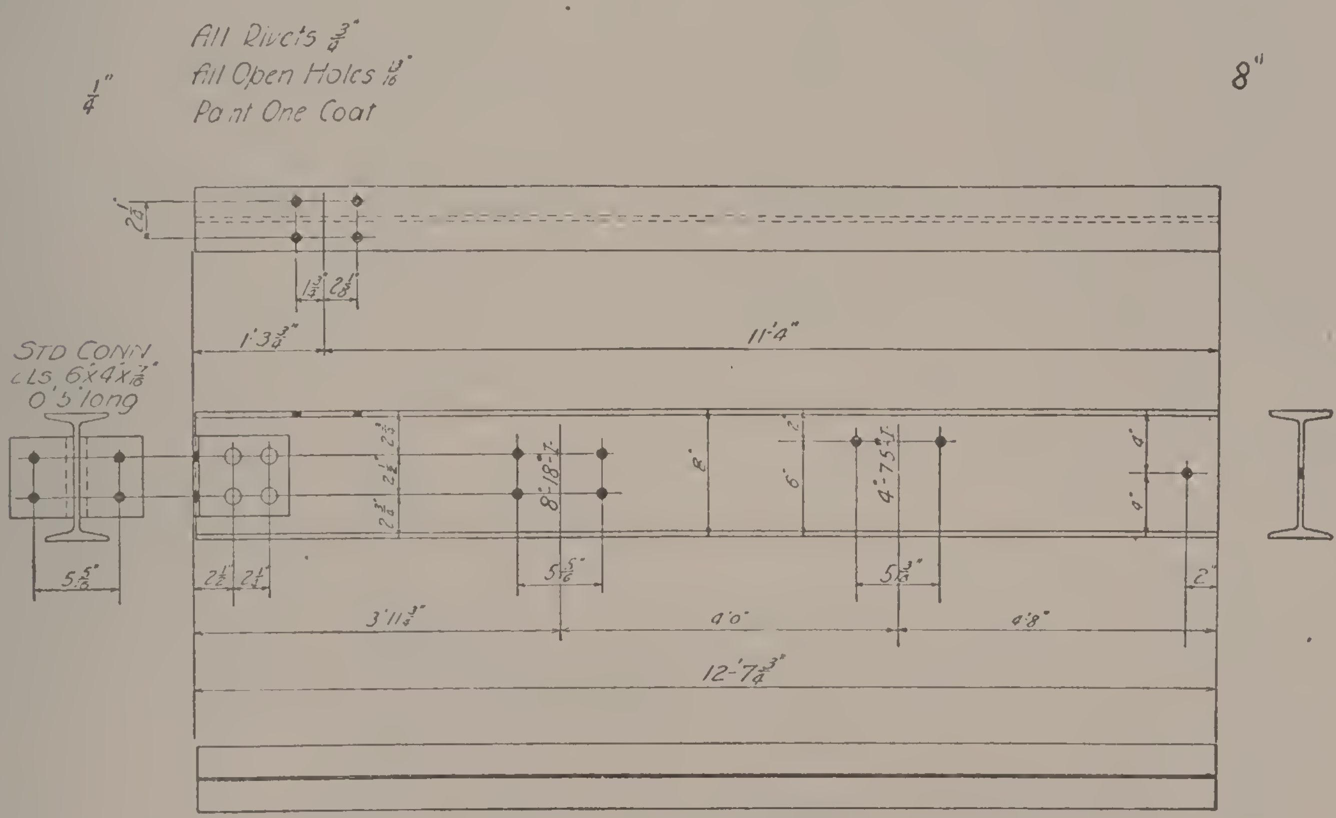
Fig. 195.

are generally spaced by an independent set of measurements from the end of the beam.

The student should note that beams cut by the mill without special directions being given are subject to a variation in length of $\frac{3}{8}$ in. under or over the length specified. If the beam rests on walls such variation is unimportant. If, however, it frames between columns and has holes connecting to the columns, such variation could not be allowed. For this reason measurements of such beams should always be marked "exact" or else at end of the sketch should be

printed "column end $\frac{1}{4}$ in. clearance". With such instructions, or similar directions in other cases to indicate how the beam rests with respect to other work, the mill will take the necessary precautions. In the case of framed beams, for instance, such notes are not necessary, as it is self-evident that no variation at these ends can be allowed.

Fig. 196 shows a beam framed into another beam, the relations of the top and bottom flanges being such as to avoid coping. Note here that it is necessary to give an end view to show the spacing of holes in the outstanding legs of connection angles. Note also the



6-8"-18" Beams-12'-7¾" lg. o.a Mark First Floor Nos. 21-26 incl.

Fig. 196.

specification as regards these angles. If the connection is standard and is placed centrally with the beam, always say "standard connection". In such cases if the shop is familiar with the standards referred to, an end view is not always necessary. If the connection is not placed centrally with the beam, or if the spacing of the holes in the legs varies any from the standard it is customary to write "standard connection, except as noted".

The first set of holes from the left-hand end in the web is for the connection of an 8-in. beam framed to this beam. Note that $5\frac{5}{16}$ in., the spacing horizontally of these holes, and $2\frac{1}{2}$ in., the spacing verti-

cally, are the measurements in the outstanding legs of the standard connection for an 8-in. I-beam.

The next set of holes in web are for the connection of a 4-in. beam which frames flush on top with the 8-in. girder; this fixes the holes at 2 in. from the top as shown.

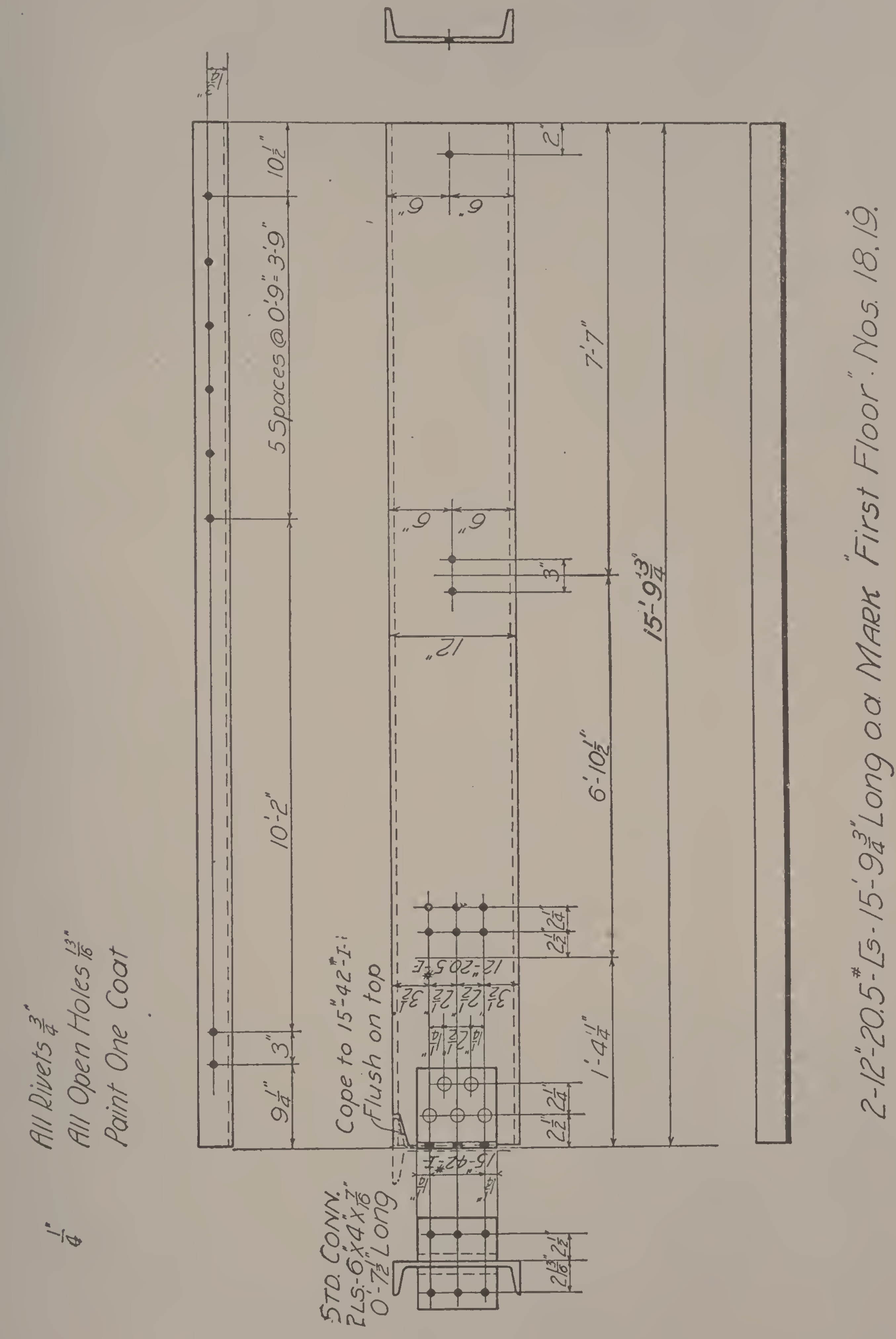
The single hole at the right-hand end is for a standard anchor rod. This measurement of 2 in. from the end is a customary measurement on such anchor holes, although some specification may call for something different.

In the flange near the left hand is shown a group of holes; these are for the connection of a channel which runs over the top of beam. As these holes are not symmetrical with regard to the axis used in locating the group, it is necessary to space each set with regard to their axis. These holes are spaced symmetrically with respect to the web of beam, and the distance between them is the standard gauge for punching the flange of an 8-in. beam. Where holes come in a flange these standard gauges should always be followed unless there are special reasons for not doing so.

In the drawing, the plan of the bottom flange is given, although there are no holes in it. Where printed forms ready for filling in measurements and details are used, this would appear and it is added here for clearness. In actual details, however, it should not be drawn if it involves extra work and if there is no punching or cuts to be shown.

Fig. 197 shows a channel detail which is similar to Fig. 196 except that it is coped. In such cases, always specify the size and weight of beam to which it is coped and give the relation of the tops or bottoms, as for instance, "cope to a 12 in., $31\frac{1}{2}$-lb. I-beam flush on bottom", or "cope to a 12-in., $31\frac{1}{2}$-lb. I-beam as shown". In case the beams do not cope flush on top or bottom, the outline of the beam to which it copes should be shown in red in the sketch, and the relations of flanges clearly indicated.

Below the sketch in beam details, is always given the specification of size and weight of beam or channel and the overall length, the number of pieces wanted and the mark to be put on them. This specification is used by the mill in entering the order for its rolling list and it is important that it agrees with the detailed measurements in the sketch. Also if the beam is cut on a bevel the extreme length



2-12" 20.5# [s. 15'-9 3/4" Long o.a. Mark "First Floor" Nos. 18, 19.
All Rivets 3/4"
All Open Holes 13/16"
Paint One Coat
5 Spaces @ 0'-9" = 3'-9"
10'-2"
7'-7"
6'-10 1/2"
1'-4 1/4"
15'-9 3/4"
12" 20.5# [
15" 42# I
Cope to 15" 42# I
Flush on top
Std. Conn.
2 Ls. 6" x 4" x 7/16"
0'-7 1/2" Long

Fig. 197.

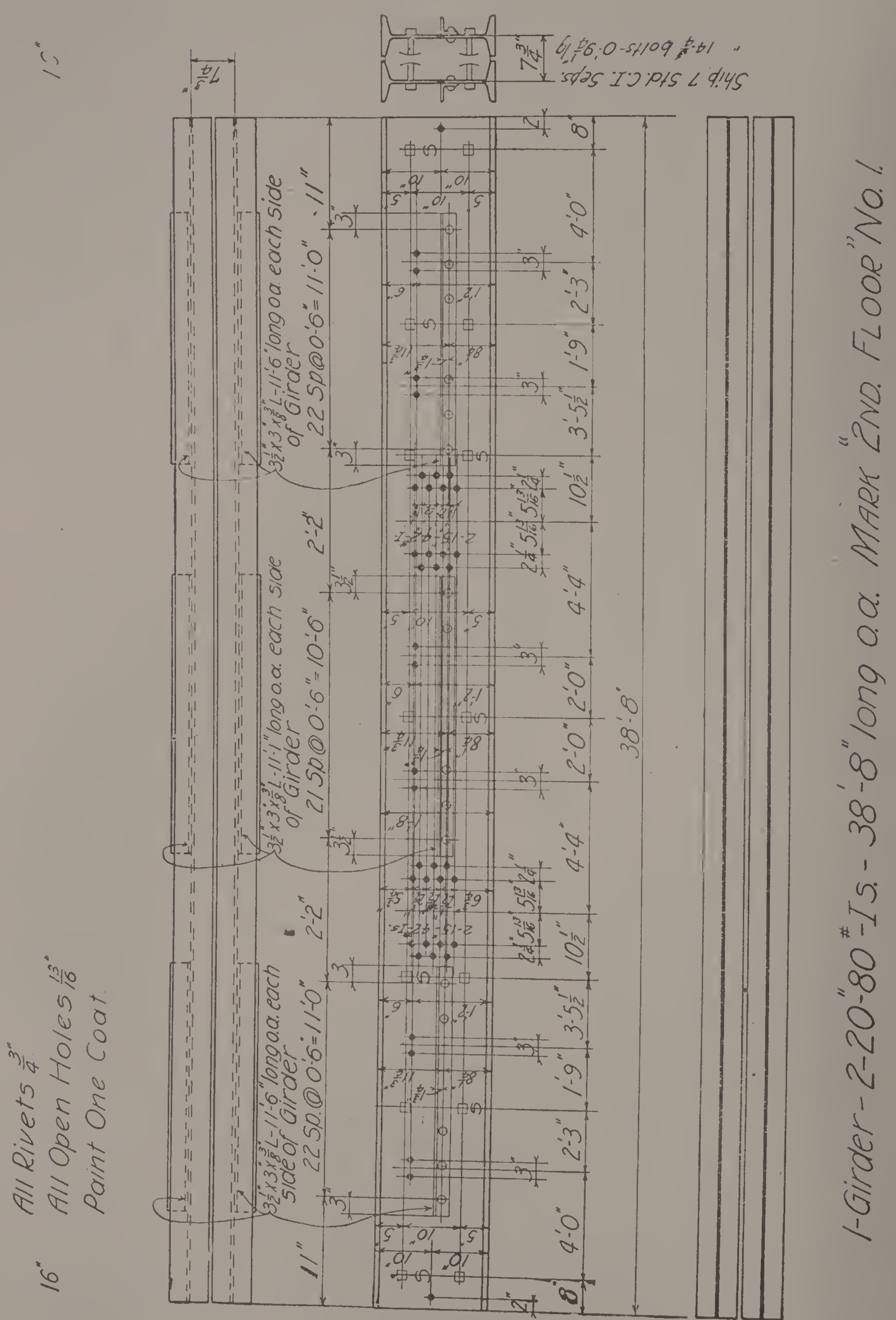

All Rivets 3/4"
All Open Holes 13/16"
Paint One Coat
Ship 7 Std. C.I. Seps.
" 14-3/4" bolts-0'-9 1/4" lg
3 1/2"x3 1/2"x3/8" L-11'-6" long o.a. each side of Girder
22 Sp.@0'-6"=11'-0"
3 1/2"x3 1/2"x3/8" L-11'-1" long o.a. each side of Girder
21 Sp.@0'-6"=10'-6"
38'-8"
1-Girder-2-20"-80#-Is.-38'-8" long o.a. Mark "2nd Floor" No. 1

Fig. 198.

of beam required to give the specified bevel should be given. Fig. 198 shows a beam girder bearing a shelf angle for the support of wind joists, or a terra cotta arch of different depth from the beam. This requires an additional line of dimensions, giving the rivet spacing and the length and position of angles. The maximum rivet pitch of six inches is generally used. Where this angle interferes with connection holes or separator bolts, as in Fig. 198, it has to be cut, and in such cases the rivet pitch must be figured out to agree with the measurements fixing the connection holes or separators.

At the top or bottom of a sheet, such general directions as apply to the work as a whole are given, as "Rivets, $\frac{3}{4}$ in. diam. except noted". "Open holes $\frac{13}{16}$ in. diam., except as noted". "Paint, one coat Superior graphite".

The student should carefully study all the dimensions in connection with the cuts, and should thoroughly understand these and the problems before starting on the subject of detailing from a plan. Note at each side of a beam sketch, are figures preceded by a plus or minus sign. These measurements denote the distance from the end of the beam in the sketch to the center of the beam or column or other member to which it connects, or the distance from face of the wall to end of the beam. These figures are not necessary for the complete detailing of the beam, but they are of great assistance in checking the drawings, as they show just how much is to be added to or substracted from the measurements on the setting plan to give the length of piece as detailed.

PROBLEMS.

1. Practice making freehand letters of the style shown on the details, both capitals and small letters. Make the letters in each word of uniform size, also practice making letters of different sizes. This is important as it is often necessary on shop drawings to put a note on a part of the drawing where space is very limited, and the writing must be small. Make a copy of the alphabet (capitals and small letters) and a copy of the numerals; also print the following in three sizes:

"All bearing plates to be faced."

One size to have a height of $\frac{3}{16}$ in. for the small letters, another size $\frac{1}{8}$ in. high, and the third size $\frac{1}{16}$ in. high.

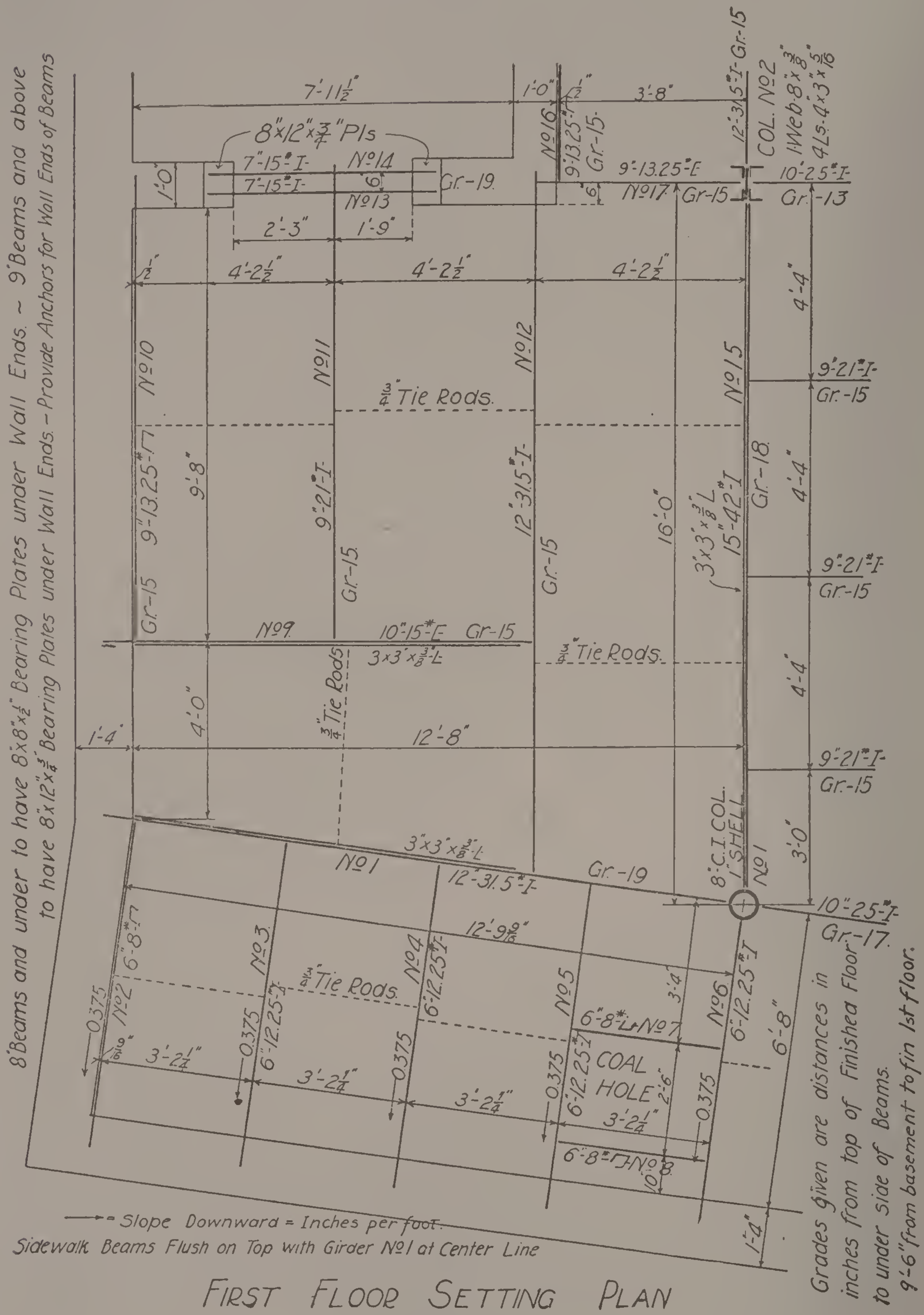
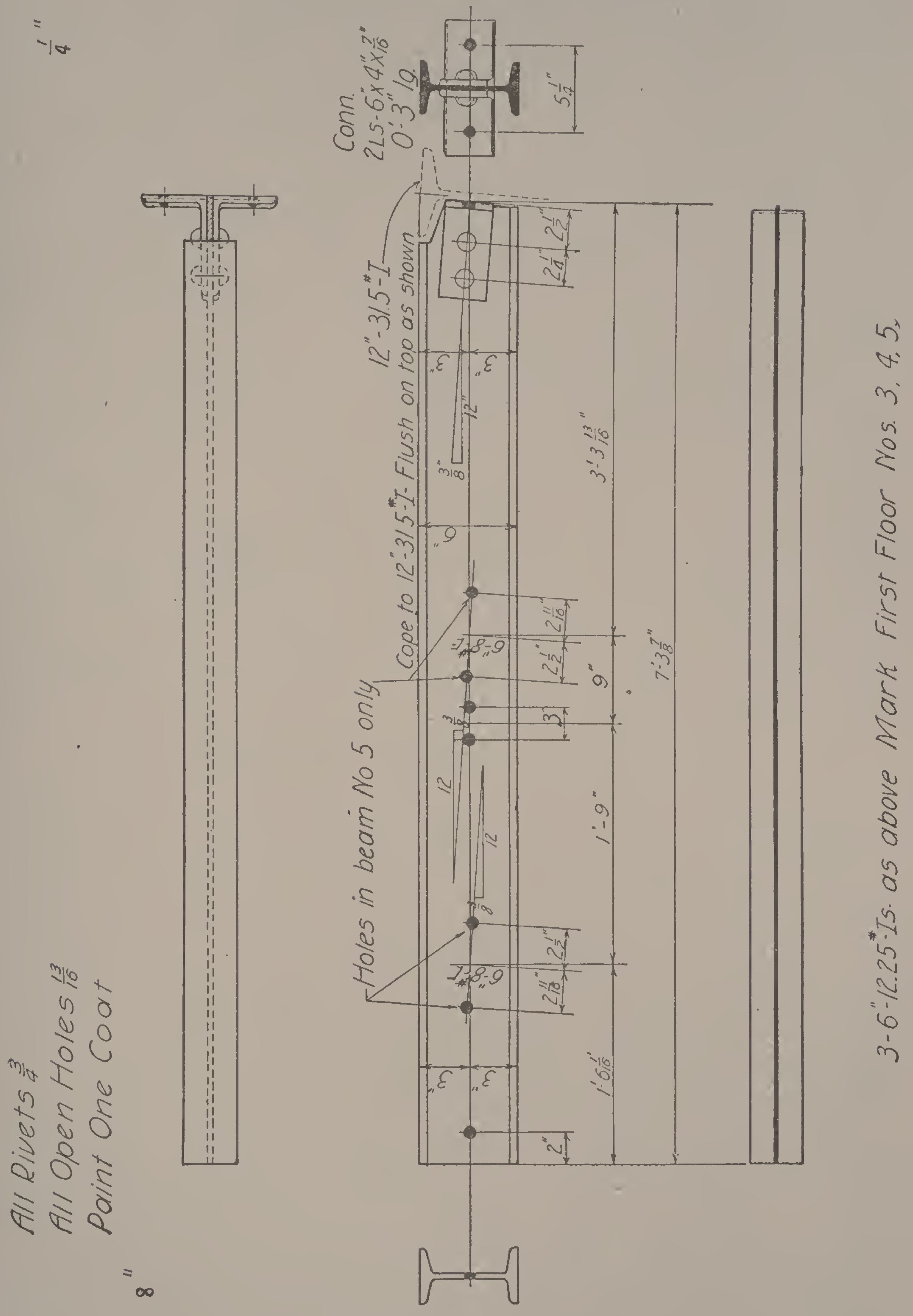

8" Beams and under to have 8"x8"x½" Bearing Plates under Wall Ends. ~ 9" Beams and above to have 8"x12"x¾" Bearing Plates under Wall Ends. - Provide Anchors for Wall Ends of Beams
8"x12"x¾" Pls
COL. No 2
No 15
No 12
No 11
No 10
No 9
No 1
¾" Tie Rods.
COAL HOLE
8" C.I. COL. 1" SHELL
Grades given are distances in inches from top of Finished Floor to under side of Beams.
9'-6" from basement to fin 1st floor.
→ = Slope Downward = Inches per foot.
Sidewalk Beams Flush on Top with Girder No 1 at Center Line
FIRST FLOOR SETTING PLAN

Fig. 199.



All Rivets 3/4"
All Open Holes 13/16"
Paint One Coat
Conn. 2Ls-6"x4"x7/16" 0'-3" lg
12"-31.5#-I
Cope to 12"-31.5#-I Flush on top as shown
Holes in beam No 5 only
3'-3 13/16"
7'-3 7/8"
3-6"-12.25#-Is as above Mark First Floor Nos. 3, 4, 5.

Fig. 200.

2. Make a shop drawing of a 6-in. I-beam, 6 ft. long, with two holes for $\frac{5}{8}$-in. rivets in top flange at each end, and $1\frac{1}{2}$ in. from the ends. Also make holes for $\frac{3}{4}$-in. rivets spaced 6 in. apart in the middle of the web for the full length. The end holes should be 3 in. from the end of the beam.

In this example, the only work specified is the punching of the rivet holes, and therefore, as no other work is required, the shop drawing will consist only of the outlines of the beam, with the rivet holes located on the same, and the spacing of the rivets shown by dimension lines, as indicated in Fig. 196.

3. Given a 20-in., 65-lb. I-beam 22 ft long, framed into a 20-in., 80-lb. I beam. The 20-in., 65-lb. I-beam has a 15-in., 42-lb. I-beam framed into each side every 5 ft. 6 in. with its top flush with the 20-in. I-beam. If the reaction of each 15-in. I-beam is 7 tons, state the number of $\frac{3}{4}$-in. rivets required for the connections of the 20-in., 65-lb. beam and for the connection of the 15-in. beam, using 9,000 pounds for shear and 18,000 pounds for bearing.

4. Make a shop detail of the 20-in., 65-lb. beam in the above problem, using standard connections.

DETAILING FROM FRAMING PLAN.

The student should now become familiar with detailing from a framing or setting plan. Fig. 199 shows such a plan upon which is all the information necessary to detail the different members. The information given on such a plan is taken from the various general plans of the building. This framing is designed for a terra cotta arch except the portion having 6-in. beams which is under a sidewalk. These beams, therefore, are on a pitch indicated by the arrows and the figures .375 which is the pitch in inches per foot.

The detail of these 6-in. beams is given in Fig. 200. Note that at the right-hand end is shown in outline the girder to which they frame, to indicate that it copes on a level with this girder. Note also that as the web of this girder is vertical while the beams pitch, the framing angles have to set on a slope with reference to the axis of the 6-in. beam, which slope is given always by a triangle of measurements, one side of which is 12 in., and the other side inches or fractions. Never use decimals for this slope on the details as the men at the shop are used to working only to inches and the nearest

sixteenth. On a plan it is well to give the slope in decimals, for if it is a fraction over or under a sixteenth, in a long slope some error might result in calculating the difference in grades unless the exact decimal was used.

The length of these beams is fixed by the measurement from the center of the girder to the face of the wall and the bearing of the beam on the wall. This bearing is generally 8 in. or more. The allowable pressures on masonry are given in Part I, and by computing the reaction on the wall, the proper bearing to give can be determined. For the smaller sizes of beams a method would give a result less than 8 in., but this should be used in such cases where possible.

The connection holes for beam No. 5 are on a pitch with reference to the axis of the beam, since the webs of beams No. 7 and No. 8 set vertically.

The tie rod holes are dimensioned on the detail but not on the plan. These holes are rarely spaced on the plan, but must be on the details. The measurements are such as to follow what is indicated by scale on the plan and avoid any other holes or connections such as connections for beam No. 7. Tie rod holes should be shown in groups of two, even if only one rod bolts to the beam, as in the case of channel No. 2.

Fig. 201 shows the detail of channel No. 9. This channel receives the ends of the terra cotta arch along the back and so it is necessary to rivet an angle on the bottom for the skewback of the arch to rest upon. Note that this stops a little short of each end in order to clear the connection angle at one end and the faces of the wall at the other. If the connection angle did not interfere, it would be well to run this as far as the flange of No. 15, and cut it to give, say $\frac{3}{8}$ in. clearance from this flange. Note the connection holes at the left-hand end for a 9-in. channel have a standard connection. Where the beam or channel is set before the brick wall is carried up, this of course can be done; if the wall is already in, it would be necessary to use an angle on one side only.

There is 1 in. from the center of the holes for the connections to the upper edge of the 3 × 3-in. angles. The connection angles for these beams come on the inside of the 16-in. channel and clearance for driving the rivet on the back is all that is required here. If the beams were framed to the back of channel, this angle would have to be cut each side of the connection.

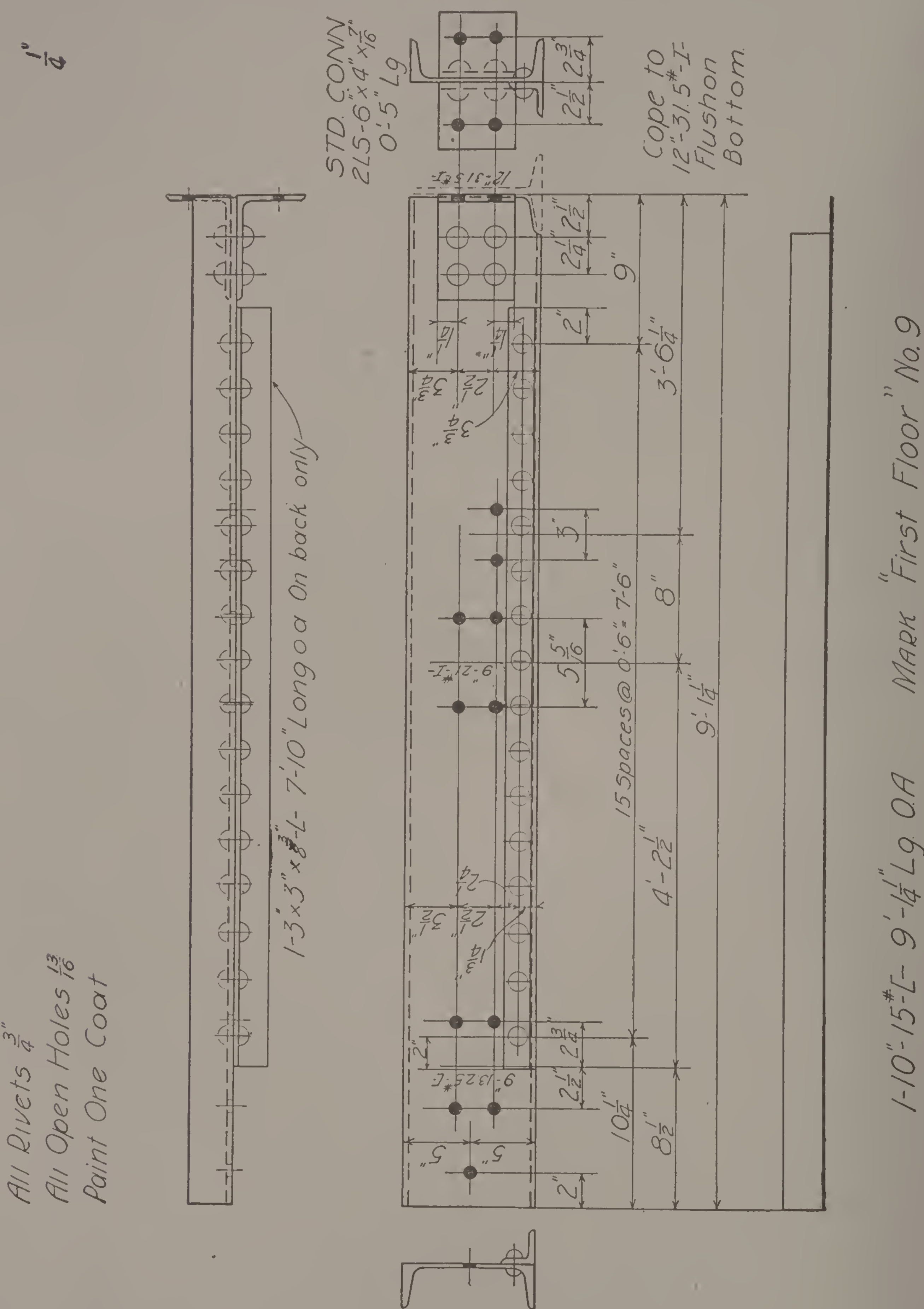

All Rivets 3/4"
All Open Holes 13/16
Paint One Coat
1-3"x3"x3/8"-L- 7'-10" Long o a On back only
STD. CONN. 2L5-6"x4"x7/16" 0'-5" Lg
Cope to 12"-31.5#-I- Flush on Bottom.
15 Spaces @ 0'-6" = 7'-6"
1-10"-15#-[- 9'-1 1/4" Lg OA
Mark "First Floor" No. 9

Fig. 201.

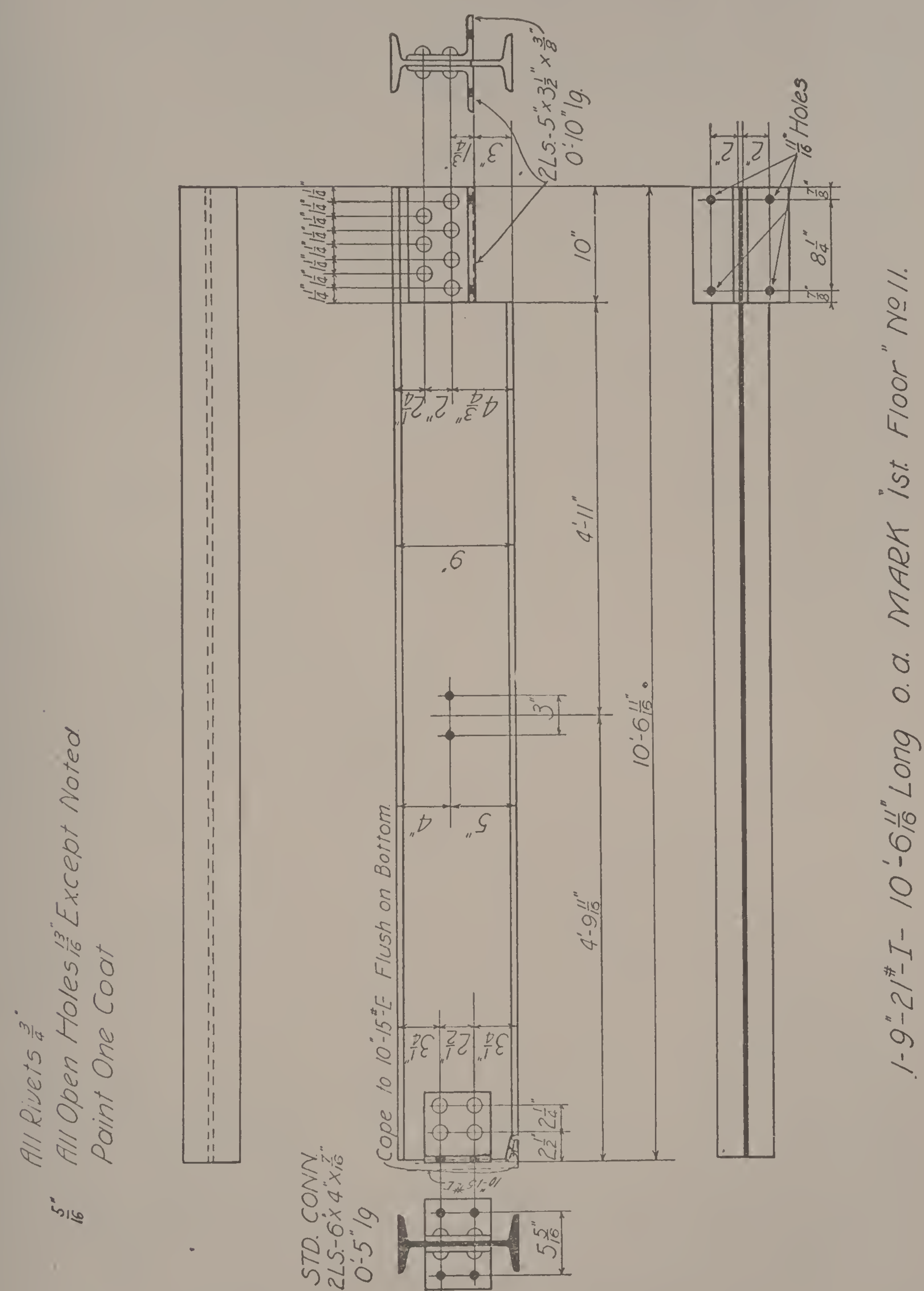

All Rivets 3/4"
All Open Holes 13/16" Except Noted
Paint One Coat
2LS.-5"×3½"×3/8"
0'-10" lg.
11/16" Holes
10"
4'-11"
10'-6 11/16"
4'-9 11/16"
Cope to 10"-15#-E Flush on Bottom
STD. CONN.
2LS.-6"×4"×7/16"
0'-5" lg
1-9"-21#-I- 10'-6 11/16" Long o.a. MARK "1st Floor" No. 11.

Fig. 202.

Fig. 202 shows the detail of beam No. 11 which frames to beam No. 9 at one end, and at the other end comes on a lintel at such a grade that the beam cannot be framed to the lintel, and owing to the small depth of the lintel, it is not possible to put a shelf angle on to receive the end of the 9-in. beam. The most practicable way, therefore, is to cut the 9-in. beam and rivet on angles which will bear directly on the top of the lintel beams. These angles have generally either a 6-in. leg or a 5-in. leg in order to contain sufficient rivets to take the reaction of the beam at this end. In this case, the cut being small as regards the depth of beam, there is sufficient web area along the inside edge of these angles to provide for the shear. If the beam had been a deeper one, and the end reaction much larger, this might not have been the case. The shear angles would then extend back to the uncut portion of the beam far enough to provide rivets to carry whole reaction to the angles, and the same number of rivets would be required in the portion over the bearing area. In general, this construction which is shown by Fig. 136, in Part II, should be followed. The holes in the horizontal legs of these angles must be spaced to agree with the holes in the flanges of the lintel beams, and are determined by the spacing of these beams and the standard gauge in the flanges. Note that $\frac{5}{8}$-in. rivets are the maximum which can be used in the flange of a 7-in. beam, and that the holes for tie rods are not in the center of the beam. The position of such holes varies; sometimes they are specified to be near the bottom of the beam. At other times where different size beams are used, as in this case, the spacing is such as to approximate the centers of all.

Fig. 203 shows the detail of the lintel beams to which beam No. 11 connects. The table on page 44, Cambria Handbook, gives the standard spacing for double beams. These spacings cannot always be followed. In this case the beams are spread more so as to bring the flanges nearer to the outside faces of the wall which rests upon them. Separators are always placed at ends over the bearings and at varying distances, center to center, as noted in Part II.

Fig. 204 shows the detail of No. 15. Observe the difference in details of two ends; one coming on the cast iron column and one on the steel column.

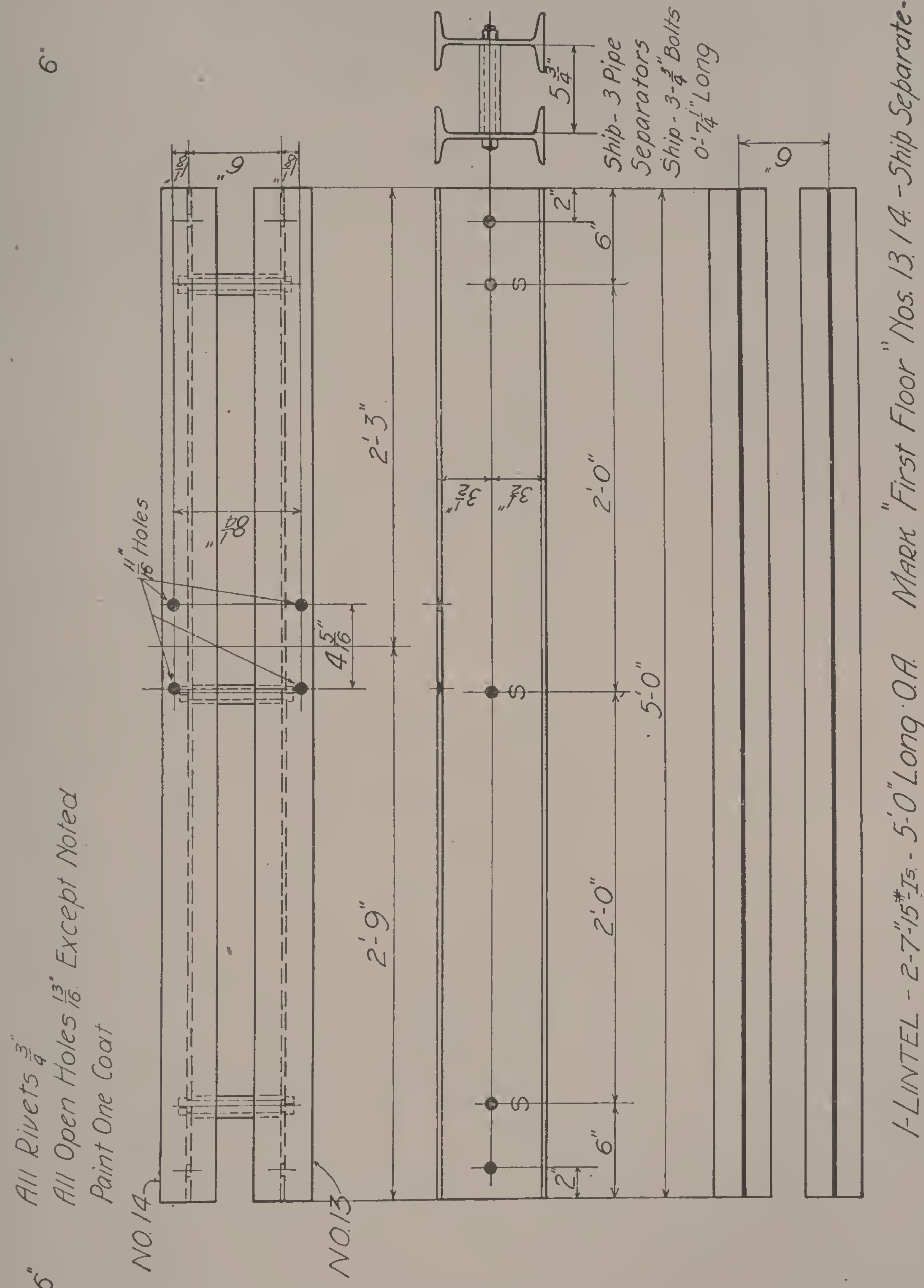

All Rivets 3/4"
All Open Holes 13/16" Except Noted
Paint One Coat
NO.14
NO.13
11/16" Holes
Ship - 3 Pipe Separators
Ship - 3-3/4" Bolts 0'-7 1/4" Long
1-LINTEL - 2-7"-15# Is. - 5'-0" Long O.A. Mark "First Floor" Nos. 13/14 - Ship Separate-

Fig. 203.

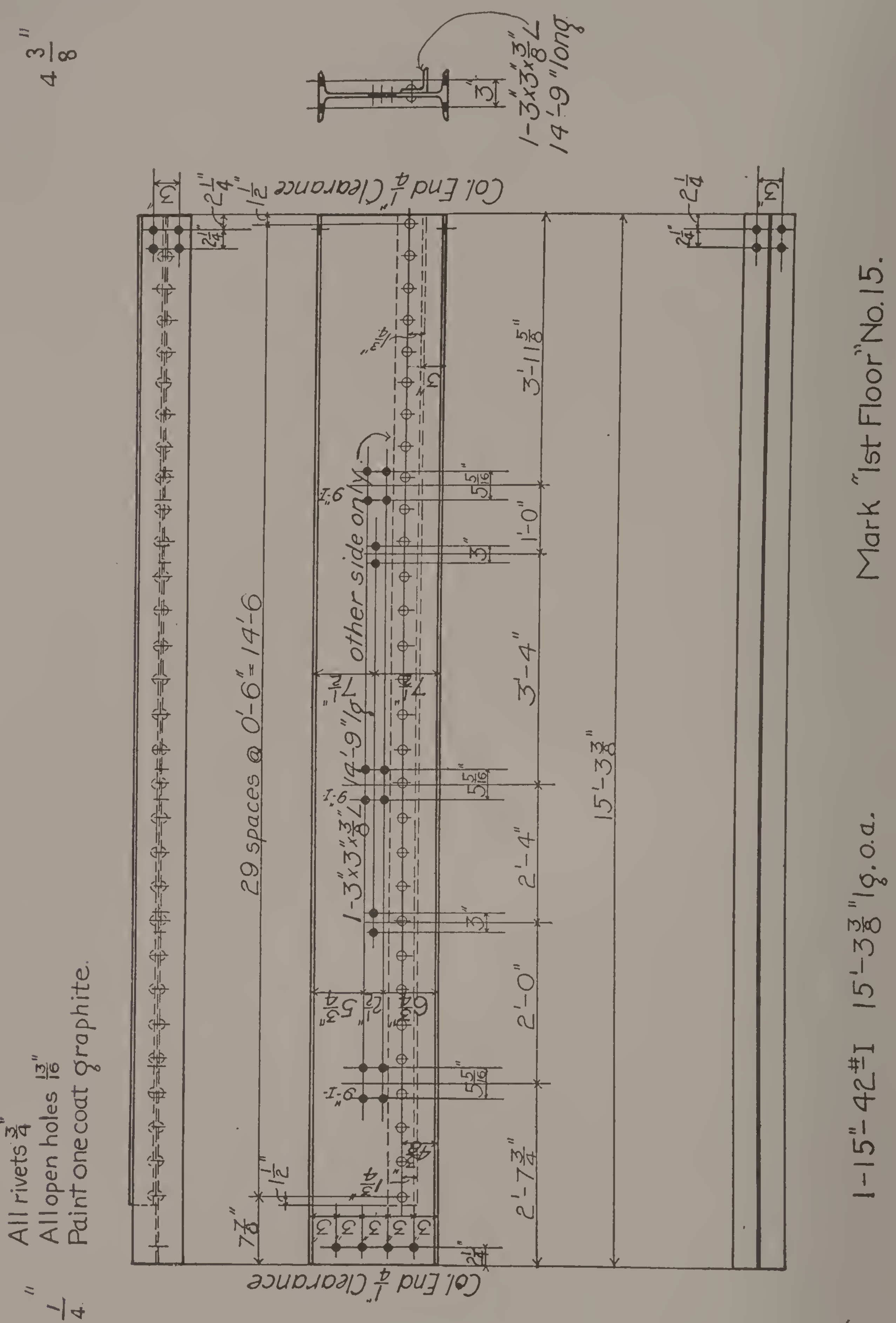

All rivets 3/4"
All open holes 13/16"
Paint one coat graphite.
1-15"-42# I 15'-3 3/8" lg. o.a.
Mark "1st Floor" No. 15.
29 spaces @ 0'-6" = 14'-6
other side only
1-3"x3"x3/8" L 14'-9" lg.
1-3"x3"x3/8" L 14'-9" long
Col. End 1/4" Clearance
Col. End 1/4" Clearance
15'-3 3/8"
3'-11 5/8"
1'-0"
3'-4"
2'-4"
2'-0"
2'-7 3/4"

Fig. 204.

As the beam is a 15-in. beam, while on one side is a 12-in. terra cotta arch, it is necessary to provide an angle on this side. The bottoms of the 9-in. beams are 3 in. above the bottom of this 15-in. girder, and if the connections were central with the 9-in. beams, the first hole would be $6\frac{1}{4}$ in. from the bottom of the 15-in. beam. In order to get clearance between this hole and the upper edge of shelf angle sufficient to drive the rivet, and to avoid cutting the angle at each connection, the shelf angle is dropped, making the upper side of the outstanding leg flush with the bottom of the 12-in. beams, and the connection on the 9-in. beams raised $\frac{1}{2}$ in.

Fig. 205 gives the detail of beam No. 12. In this case, the length of beam cannot be obtained directly from the framing plan, as the beam No. 1 is not perpendicular to beam No. 12. The difference in measurement of the ends of No. 1 from the wall line is 1 ft. 10 in., and the length parallel to this wall and square with beam No. 12 is 12 ft. 8 in. from the center of the column. As No. 12 is 4 ft. $2\frac{1}{2}$ in. from the center of the column, the bevel from the column to No. 12 is $\frac{4.21}{12.67} \times 22 = 7.31$ inches, or $7\frac{5}{16}$ in., to the nearest sixteenth.

The length of No. 12 from the face of wall to center of No. 1 on this line, therefore, is 14 ft. $10\frac{11}{16}$ in. The bearing on wall being 8 in., and the clearance at the other end $\frac{1}{4}$ in., the total length of beam is 15 ft. $6\frac{7}{16}$ in.

The girder No. 1 coming under the sidewalk is 4 in. lower than beam No. 12. This is not enough to get a shelf angle on the girder, or to get angles over the top of the girder, as in the case of beam No. 11. It is necessary, therefore, to drop the connection on No. 12 and notch the beam over the top flange of No. 1. This notching is not figured on as reducing the required number of rivets in the connection, but does give an added element of strength and of stiffness. In order to get the connection in, it is necessary to go within 1 in. of the bottom flange of No. 12 and the top flange of No. 1; thus encroaching somewhat on the fillet in each case. As the required number of rivets can not be obtained in two lines, as is generally the case, it is necessary to use special spacing as shown. The connection specifies bent plates rather than angles; where the bevel is over 1 in. to the foot, it is customary to use plates.

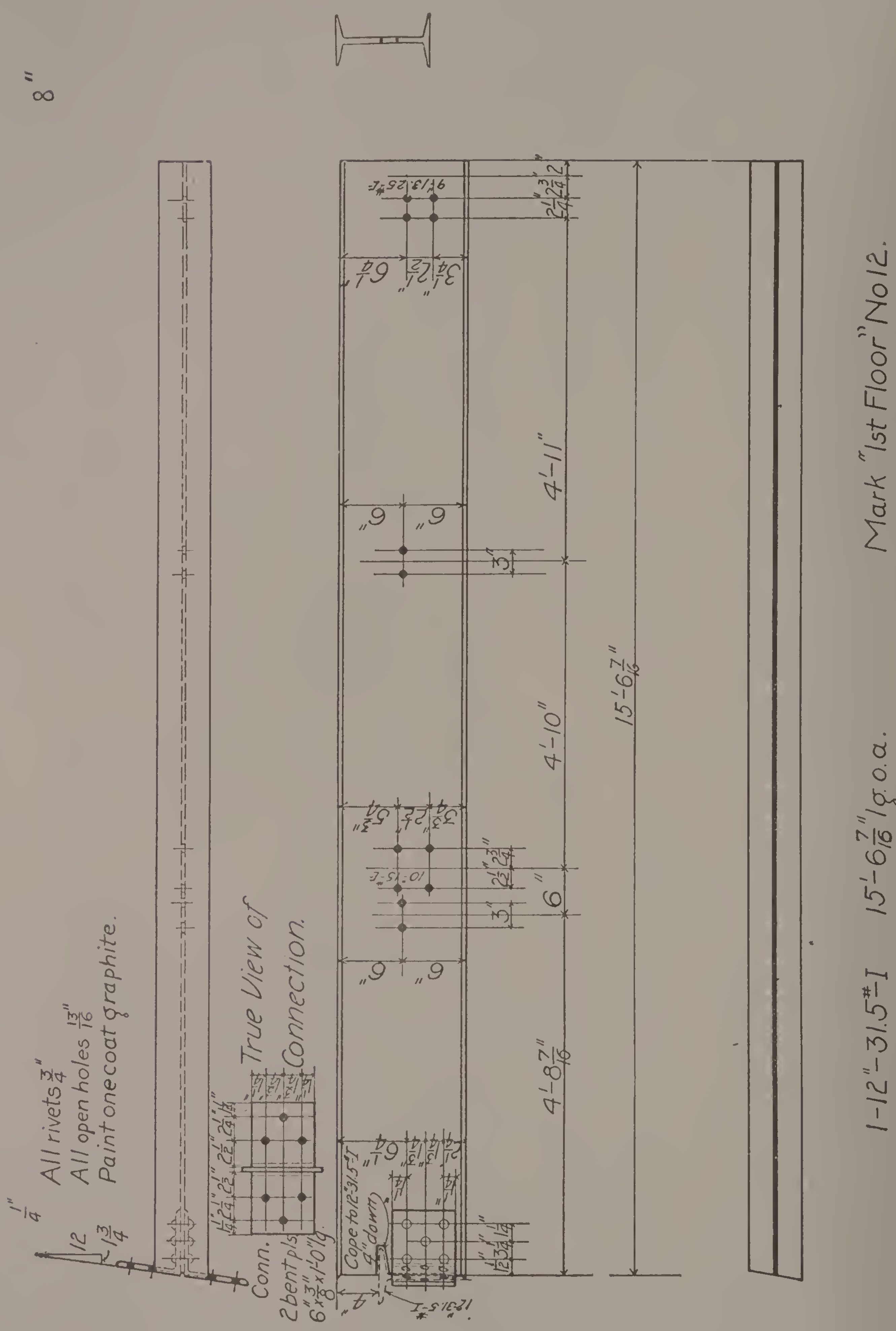

All rivets 3/4"
All open holes 13/16"
Paint one coat graphite.
True View of Connection.
Mark "1st Floor" No 12.
1-12"-31.5#-I 15'-6 7/16" lg. o.a.
15'-6 7/16"
4'-11"
4'-10"
4'-8 7/16"

Fig. 205.

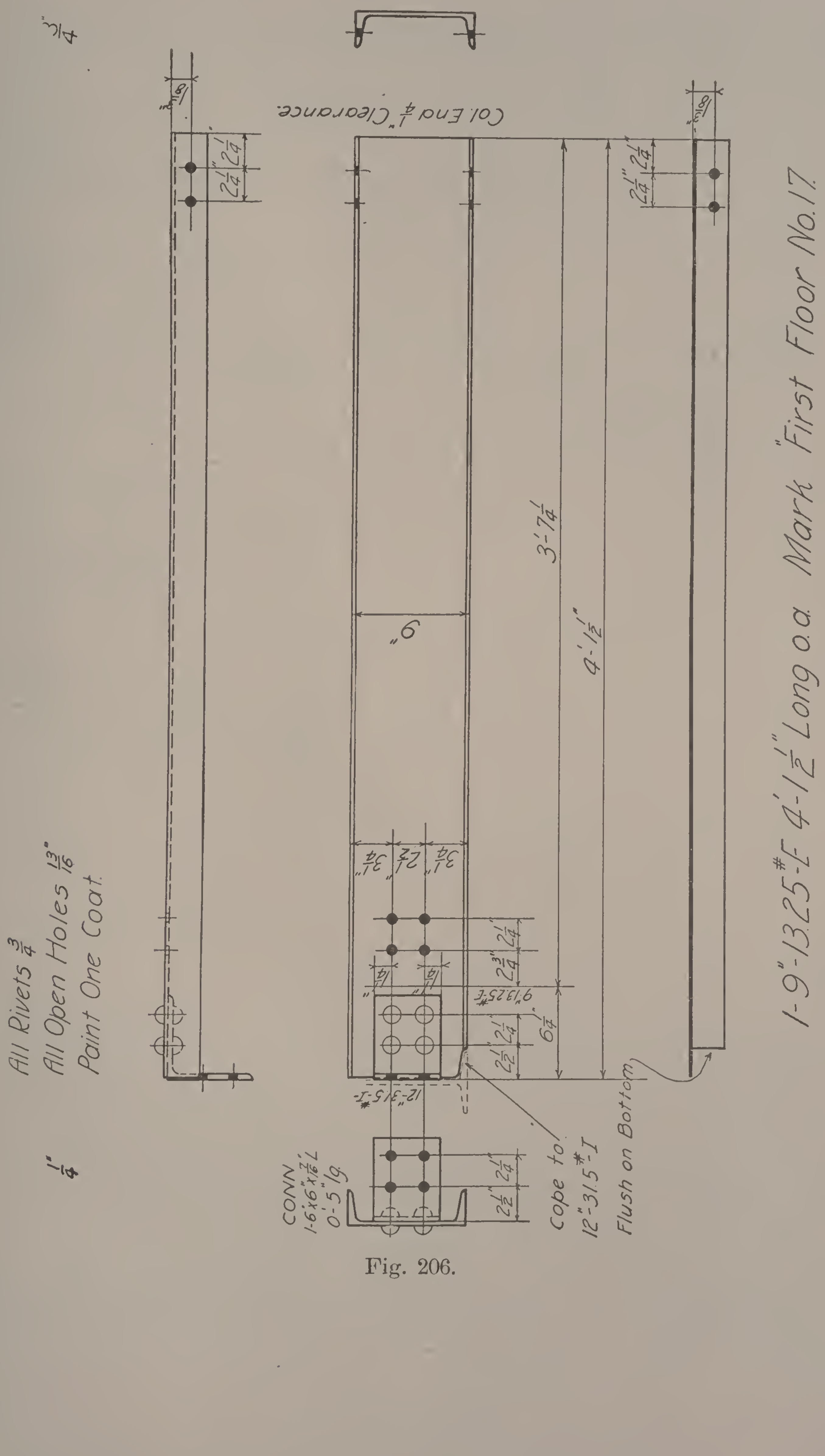
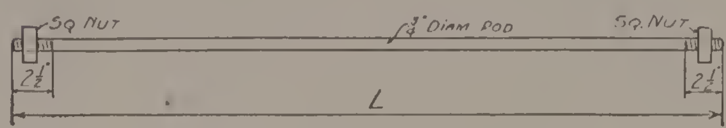

All Rivets 3/4
All Open Holes 13/16"
Paint One Coat.
1-9"-13.25#-I 4'-1 1/2" Long o.a. Mark "First Floor No.17"
Col. End 1/4" Clearance.
Cope to 12"-31.5#-I
Flush on Bottom
CONN 1-6x6x7/16 L 0-5" lg

Fig. 206.

Fig. 206 gives the detail of channel No. 17. This channel has a single angle framing. This is a case where the channel comes into a wall so that a connection angle on the back side cannot be reached. In order to get the necessary number of rivets in the outstanding leg, therefore, a 6 × 6-in. angle must be used. The holes at the left-hand end in the web are for the connection of a channel similar to what is shown in the end view.



SCHEDULE OF TIE RODS

No. of PIECES	LENGTH "L" FEET	LENGTH "L" INCHES	MARK
3	3	5¼	FIRST FLOOR
4	4	5½	" "
1	4	11½	" "

SCHEDULE OF FIELD BOLTS
for FIRST FLOOR

NO of PIECES	SIZE	LENGTH FEET	LENGTH INCHES
29	¾	0	1¾
47	¾	0	2
4	⅝	0	1¾

SCHEDULE OF BEARING PLATES
for FIRST FLOOR

NO. of PIECES	SIZE	LENGTH FEET	LENGTH INCHES
5	8" × ½"	0	8
6	8" × ¾"	1	0

Fig. 207.

These holes are located from a line which in turn is located from the end of the channel; this axis is the back of the channel framing in.

Fig. 207 gives a schedule of tie rods and of field bolts, and of bearing plates for the framing as shown on Fig. 199.

Note the over-all lengths of the tie rods is 3 in. longer than the length, center to center of beams. This allows $1\frac{1}{2}$ in. for the two nuts, about $\frac{3}{8}$ in. for half the thickness of the two webs and about $\frac{9}{16}$ in. projection of rod beyond the nut.

The length of field bolts is always given from the underside of the head to the end of the bolts. The grip is the thickness of the metal between the underside of the head and the nut; that is, the thickness of the connection angles and the web. A projection of $\frac{1}{4}$ in. or $\frac{1}{2}$ in. beyond nut should be allowed for.

Fig. 208 shows the setting plan of another floor, a part of which the student will be required to detail as problems.

Fig. 209 shows the detail of the beam girders, Nos. 2 and 3. As the beams are spaced close together, connections can be used only on the outside of the webs. The same number of rivets in the out-

standing legs must, of course, be used, as would be required for a double-angle connection, and more rivets must be used through the web, as these are in single shear instead of bearing or double shear. In the case of the beams shown, seven rivets are all that are necessary, although the standard connection requires eight.

In such a connection as girder No. 2 to girder No. 1, it is necessary to use bolts, as there is no way of riveting. In the case of the connections of beams to girders Nos. 2 and 3, rivets might be used

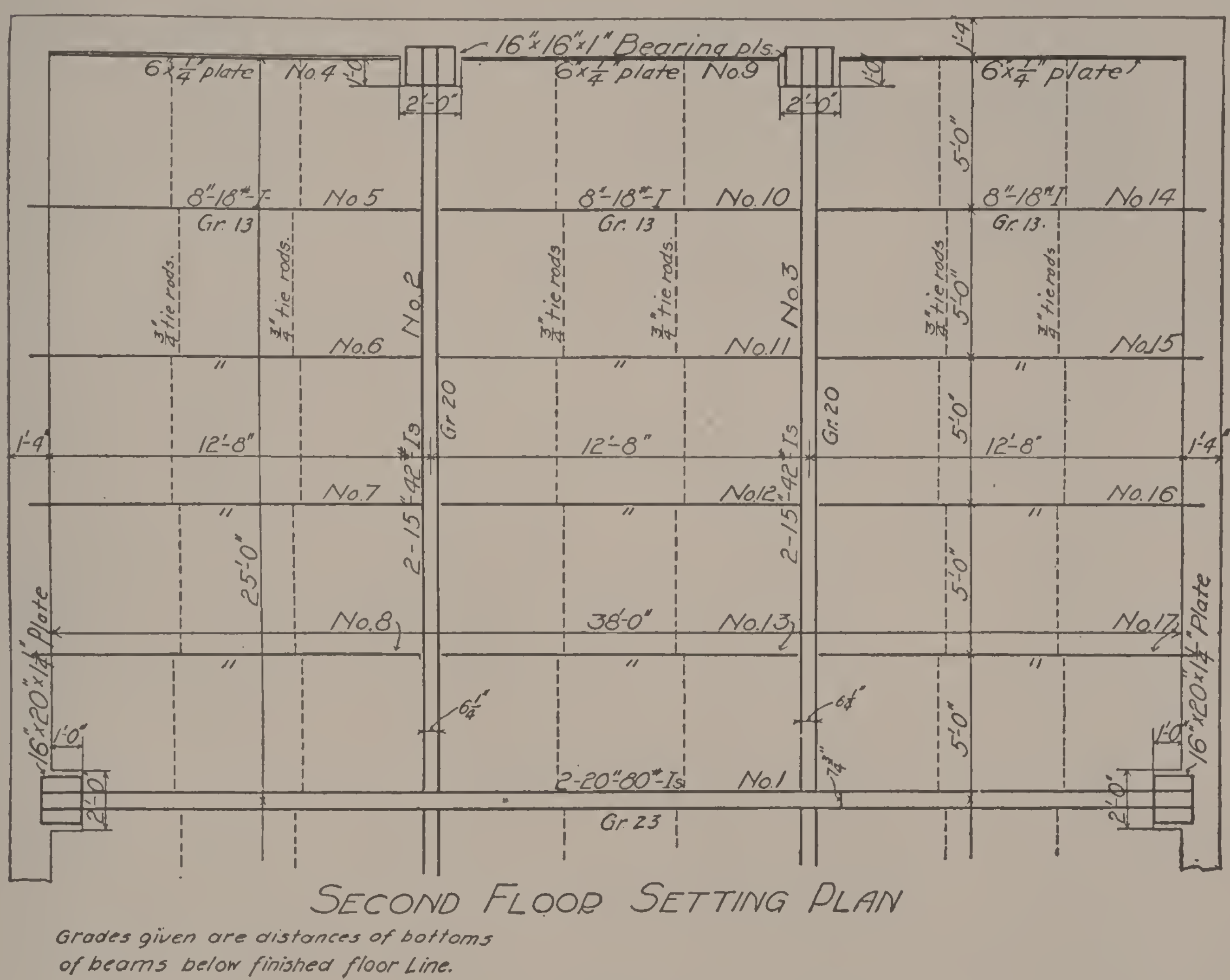


Fig. 208.

by separating the two beams forming each girder and sliding the framing of each outside bay over on the wall far enough to get in between the beams of girders to hold the rivets. After all the beams had been riveted up, the whole frame could then be moved back into position, and the girders bolted up. Such an operation would be expensive, as it would require considerable extra moving of the beams. In general, bolts through webs of both beams would be used. If the connection was very heavy or the greatest possible number of bolts barely sufficient for the load, turned bolts should be used. In this

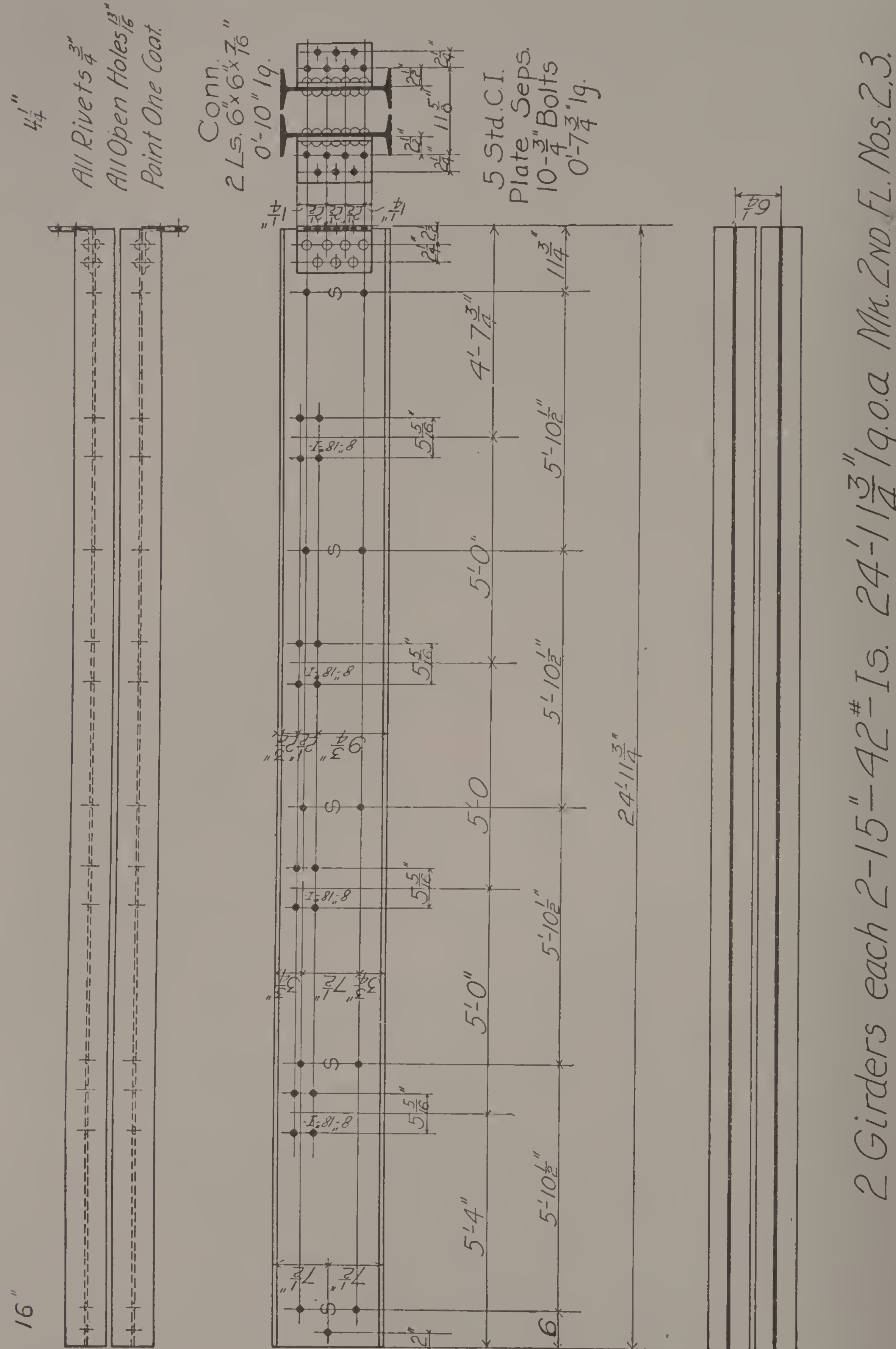

All Rivets 3/4"
All Open Holes 13/16"
Paint One Coat.
Conn. 2 Ls. 6"x6"x7/16" 0'-10" lg.
5 Std. C.I. Plate Seps. 10-3/4" Bolts 0'-7 3/4" lg.
2 Girders each 2-15"-42#-Is. 24'-11 3/4" lg. o.a. Mk. 2ND Fl. Nos. 2,3.

Fig. 209.

case, the holes should be punched $\frac{1}{16}$ in. smaller than the diameter of the rivet, and then reamed to a diameter $\frac{1}{16}$ in. larger than the rivet so as to remove all ragged edges; the bolts would be turned down to a true diameter, the exact size of holes, for their whole length.

Fig. 198 shows the detail of girder No. 1. This girder receives a terra cotta arch on each side and as the girder beams are deeper than the floor beams, angles must be used to receive the arch. These angles have to be cut to clear the connection angles on the beams framing in, however. The separators must be spaced so as not to interfere with the rivets in the shelf angles.

The student should carefully study every detail shown in the preceding cuts, and should thoroughly understand every feature of them and every note, and the reason for all the special features appearing in them. He should work out for himself all the measurements given by the details so that he will understand these and know just how to proceed in other cases.

PROBLEMS.

1. Make a shop detail of a 10-in., 25-lb. beam, 12 ft. long, resting 8 in. on a brick wall at each end and having holes for anchors at each end, and holes for tie rods in the center.

2. Make a shop detail of a 12-in., 40-lb. beam, 15 ft. long, framing into a 15-in., 42-lb. beam flush on bottom at one end and into an 18-in., 55-lb. beam 1 in. below the top at the other end. The 12-in. beam has holes for three 8-in., 18-lb. beams with standard connections spaced equally throughout the length, center to center, between girders.

3. Make a shop detail of a 9-in., 21-lb. beam with a $4 \times 3 \times \frac{3}{8}$-in. angle riveted to the beam the full length. This angle to be placed with the horizontal leg down and as near the bottom of the 9-in. beam as possible, and the 4-in. leg to be out. The beam rests on a wall 8 in. at each end and it is 13 ft. 9 in. between walls.

4. Make a detail covering channels No. 7 and No. 8, shown in Fig. 199.

5. Make a detail of channel No. 17 in Fig. 199.

6. Make a detail of channel No. 10 in Fig. 199.

7. Make details covering the 5 to 8-in. beams, and the 14 to 17-in. beams in Fig. 208.

COLUMN DETAILS.

There are five main features in the detailing of a column.

1. The base or foot of the column.
2. The shaft or the line members composing the column.
3. The cap or top of the column.
4. The connections for other members to the column.
5. The bill of material required to make up the completed column.

A column detail is of necessity more complicated than a beam detail and may at first appear so confused as to be unintelligible. If the student will bear in mind, however, these five features and take each by itself, it will soon become clear.

Details of Base. The character of the base or foot of the column depends upon what it rests. If this is the first section of the column, it will generally rest on a cast iron ribbed base, or a plain steel or cast iron plate. It is the duty of the designer and not of the draftsman to determine which one of these will be used.

Fig. 224 shows a detail of a foot of a column resting on a cast iron ribbed base. The base is always designed so as to take the load of the column by direct bearing between the line members and the top of the base, and the angles which are riveted to the column are intended simply to hold it in position in the base.

If a plain cast iron plate is used, a connection similar to the above would generally be used, because in this case the load would be light and the plate thick enough to withstand the upward pressure without spreading the foot of the column. Such plates must be calculated in the same way explained for bearing plates under beams. See Part II, page 96. The projection of the plate beyond the shaft is exposed to bending just as the plate under a beam is where it projects beyond the flange.

If a steel base plate is used, this is generally riveted to the column and the load then must be spread out beyond the lines of the shaft by vertical plates or angles, called shear plates or angles, so as to avoid an excessive bending moment. The size and shape of this plate are determined by the area required to properly distribute the load on the masonry and the direction in which the foot can be most readily spread by means of the shear plates and angles. The

thickness of the plate is determined by the same formula as before used for cast iron and bearing plates; generally it is $\frac{3}{4}$ or 1 in. thick. The projection is the distance beyond the edge of the shear plate, or the outstanding leg of the shear angle.

The number of rivets between the column and the shear plate or angle is determined by considering the area exposed to bending, as the outer edges of the base plate and of the shear plate. The load being uniformly distributed, the pressure per square inch is the total load divided by the total area of the base plate, and the load on rivets in the shear plate, therefore, is this unit pressure multiplied by the area over which the shear plate distributes it, as above stated. The balance of the column load may be considered as distributed by direct bearing of the line members on the plate.

It is generally not necessary to use more than six rivets in one line for connection of shear plates, and some system of plates and shear angles should be used so as not to exceed this number, or if this is not possible, a cast iron ribbed base, or a smaller steel plate bearing on steel beams should be used. The exact number of rivets determined as above may be decreased somewhat if this exceeds six, as the plate, even if not supported by the angles or shear plate, is capable of taking some of the load before bending would result. Judgment determines largely how much consideration can be given to this factor.

If the column is an upper section, and rests on the top of another section, the foot is then generally of a character similar to what is shown in Fig. 214. It is, of course, essential that the holes in the foot should match the holes previously detailed in the cap of the lower section. Where a horizontal splice plate is used, this should be large enough to bear over all the line members. Where the column below is of greater dimension, the fillers must be shipped bolted to the foot of the column.

Cap Details. These are of the general form shown in Figs. 211 and 214. They will vary somewhat according to the sections composing the column. In high buildings it is essential to have vertical splice plates to give the necessary stiffness to the joint. Usually this splice plate extends far enough up to take three lines of rivets. The ends of the columns are always faced to true plates at right angles with the axis of the column, and so the splice plate is not designed to transmit any of the vertical load.

In arranging the holes in the cap, it is necessary to consider the section which comes above so as to space these holes to conform to what may be feasible in the foot of the upper section. This other section may be of smaller dimensions, and it may then be necessary to space the holes in the lower section closer, so as to make it possible to rivet up without interfering with the line members, or coming too near the edge of the connection angle.

Shaft Details. This consists in locating all shelf and bracket angles and connection holes, or other special connections, and in spacing the rivets so as to conform to these connections, and not to exceed the maximum or minimum distance.

The rivets in shelves and brackets having been spaced, and the position of these on the shaft from the top and bottom having been fixed, it only remains to divide the space into as many equal rivet spaces as possible, and put the odd spaces near the top or bottom of the shaft.

Six inches is the maximum pitch allowed, and if the metal through which the rivet goes is less than $\frac{3}{8}$ in. thick, the maximum pitch is sixteen times this thickness. Three times the diameter of the rivet is the minimum pitch which can be used.

Illustrations of Column Details. In making column details, the views are not complete views, regarded as mechanical drawings. The essential feature is clearness and, as the drawing must of necessity show as many details, it is important to omit what is not necessary. For instance, a column which is made up of four angles and a web plate should show, to be complete, the dotted lines indicating the legs of the angles riveted to the web. It adds to the clearness, however, to omit these where a connection comes on the flange. Similarly, in showing a view of the flange, it will add to the clearness to omit showing the connection angles which rivet to the web and are sometimes indicated back of the flange by dotted lines.

In the case of the web view, it is generally necessary to show what is on both sides of the web, as except in special cases, one elevation only of the web is given.

Fig. 210 gives the detail of the cast iron column shown on the setting plan in Fig. 199. The foot of the column rests on a solid cast iron plate and sets into a ring on this plate to prevent lateral movement. There are a variety of details for holding the foot of

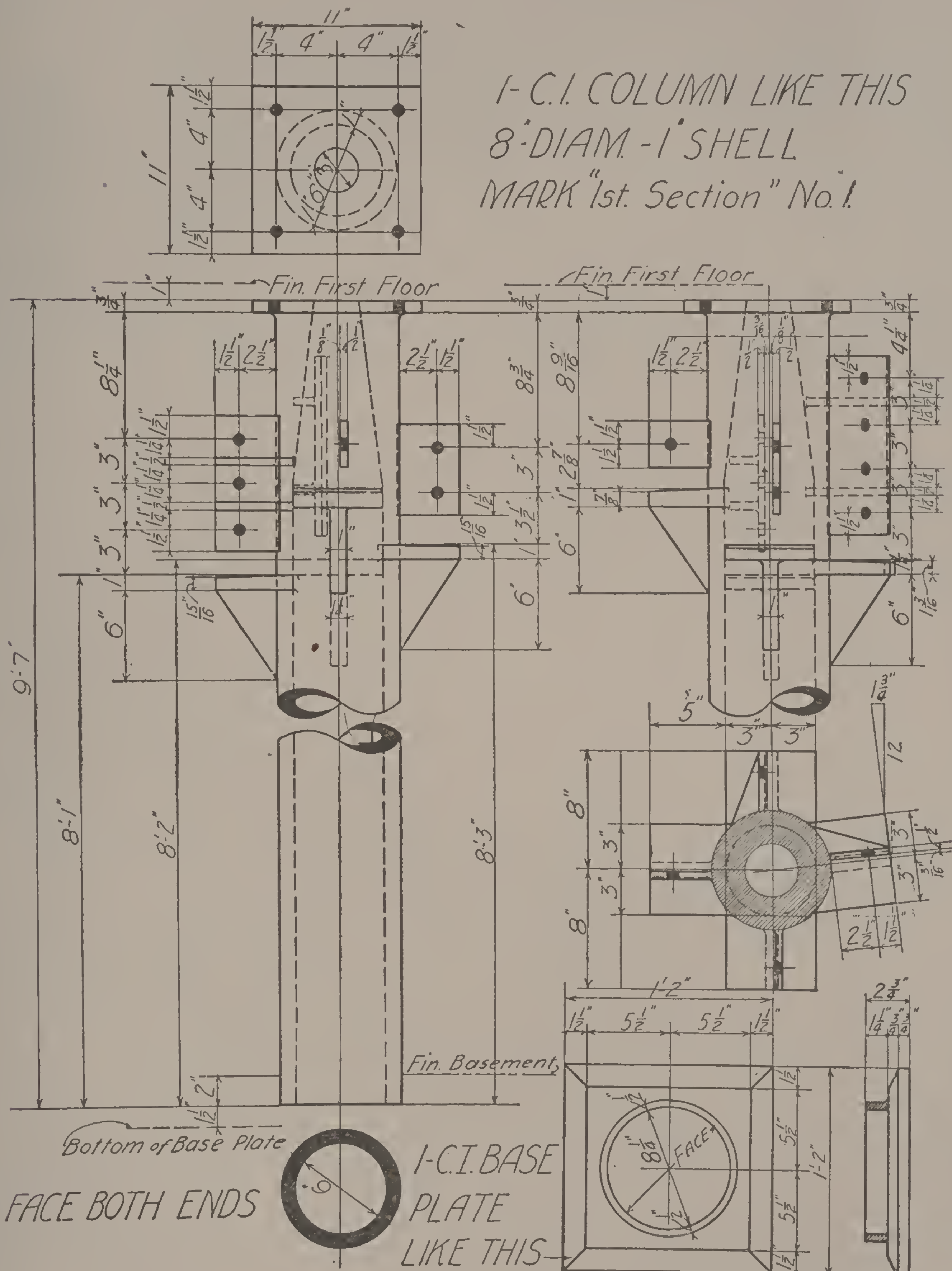
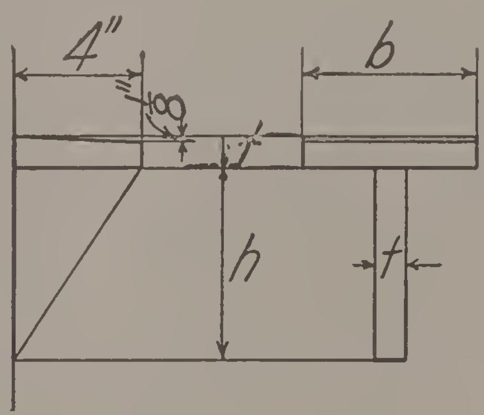

1-C.I. COLUMN LIKE THIS
8"DIAM.-1" SHELL
MARK "1st. Section" No. 1.
Fin. First Floor
Fin. First Floor
Fin. Basement
Bottom of Base Plate
FACE BOTH ENDS
1-C.I. BASE
PLATE
LIKE THIS
FACE
9'-7"
8'-1"
8'-2"
8'-3"

Fig. 210.

the column in place, but this is one very generally used. The relation of the bottom of the base plate to the finished floor line should always be given to enable the plate to be set at the proper grade.

Connection of beams to columns is by a shelf under the beam and a lug bolted to the web to hold the beam in position. The top surface of the bracket should slope about $\frac{1}{16}$ in. so as to avoid the tendency of the beam when it deflects to bring the load on the outer edge of the bracket.

The lugs are generally $\frac{1}{2}$ or $\frac{3}{4}$ in. thick. The bracket should of course be wide enough to receive the flange of the beam. The thickness of the bracket and rib under it varies with the load. This rib in general is beveled at an angle of 30 degrees with the axis of the column. The accompanying table gives the thicknesses which are sufficient for most cases.

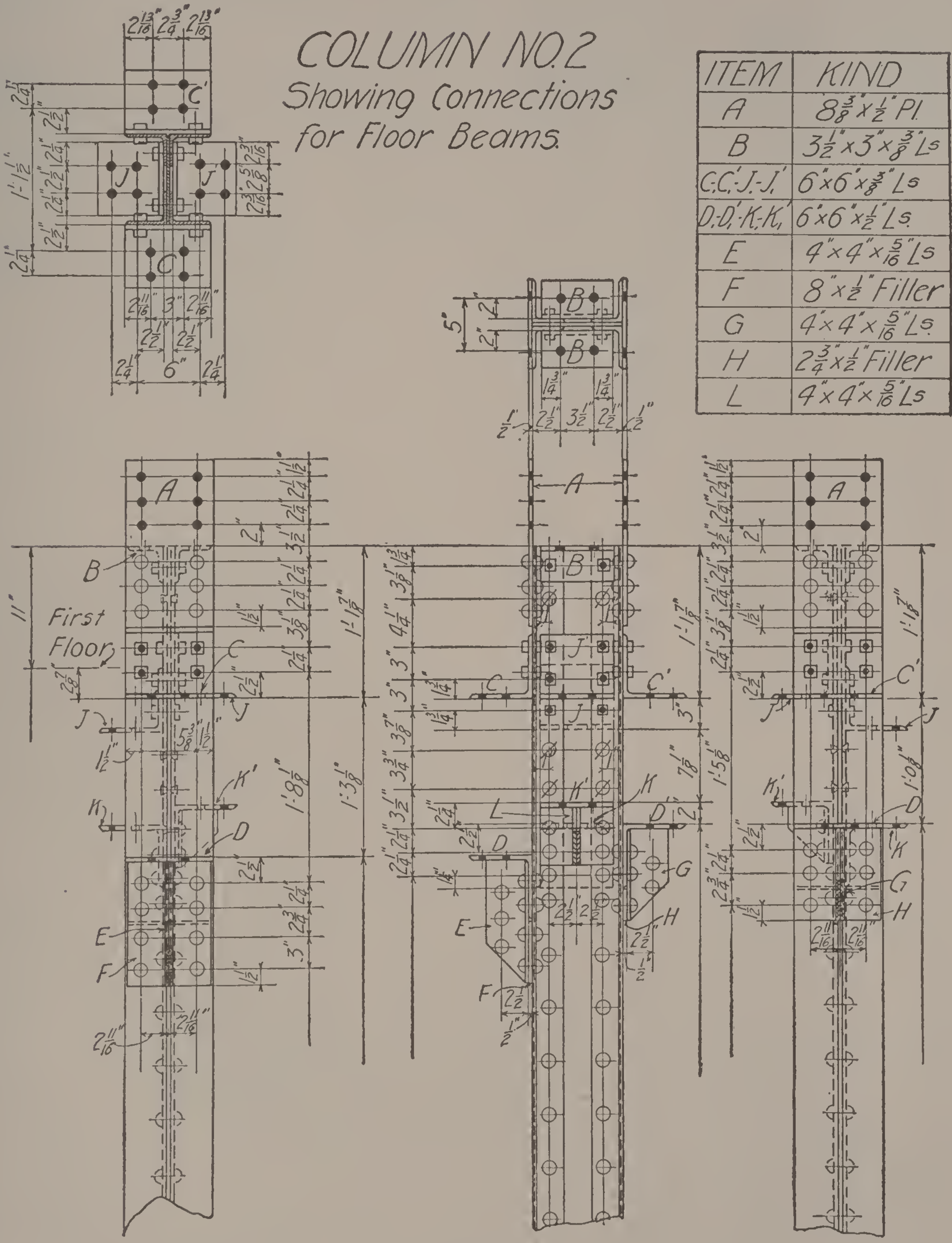


Size of Beam	-b-	-t-	-h-
Up to 7"	4"	$\frac{3}{4}$"	6"
7",8",9",10",12"	$5\frac{1}{2}$"	1"	6"
15"	6"	$1\frac{1}{4}$"	6"
18",20"	$6\frac{1}{2}$"	$1\frac{1}{4}$"	8"
24"	$7\frac{1}{2}$"	$1\frac{1}{2}$"	9"

Dimensions of Brackets and Lugs.

The lugs are braced by ribs back to the column shaft so as to prevent being broken off. The flange at the top which connects the two sections of the columns may be $\frac{3}{4}$ in. or more, up to $1\frac{1}{2}$ in. in some cases; for usual sizes of columns, $\frac{3}{4}$ or 1 in. is sufficient. The holes in the flange must be spaced so as to enable bolts to be turned up without interfering with the shaft of the column and the distance from the top of the beam to underside of the flange must be sufficient for this purpose.

Fig. 211 gives the details of the beam connections, and the cap for column No. 2 in Fig. 199. This is not a complete shop detail but shows one of the steps in the complete detailing of a column which is generally the first step; namely, the drawing of connections, locating the same on the shaft and spacing rivets in the connections.



ITEM	KIND
A	$8\frac{3}{8}''\times\frac{1}{2}''$ Pl.
B	$3\frac{1}{2}''\times3''\times\frac{3}{8}''$ Ls
C, C', J, J'	$6''\times6''\times\frac{3}{8}''$ Ls
D, D', K, K'	$6''\times6''\times\frac{1}{2}''$ Ls.
E	$4''\times4''\times\frac{5}{16}''$ Ls
F	$8''\times\frac{1}{2}''$ Filler
G	$4''\times4''\times\frac{5}{16}''$ Ls.
H	$2\frac{3}{4}''\times\frac{1}{2}''$ Filler
L	$4''\times4''\times\frac{5}{16}''$ Ls

Fig 211.

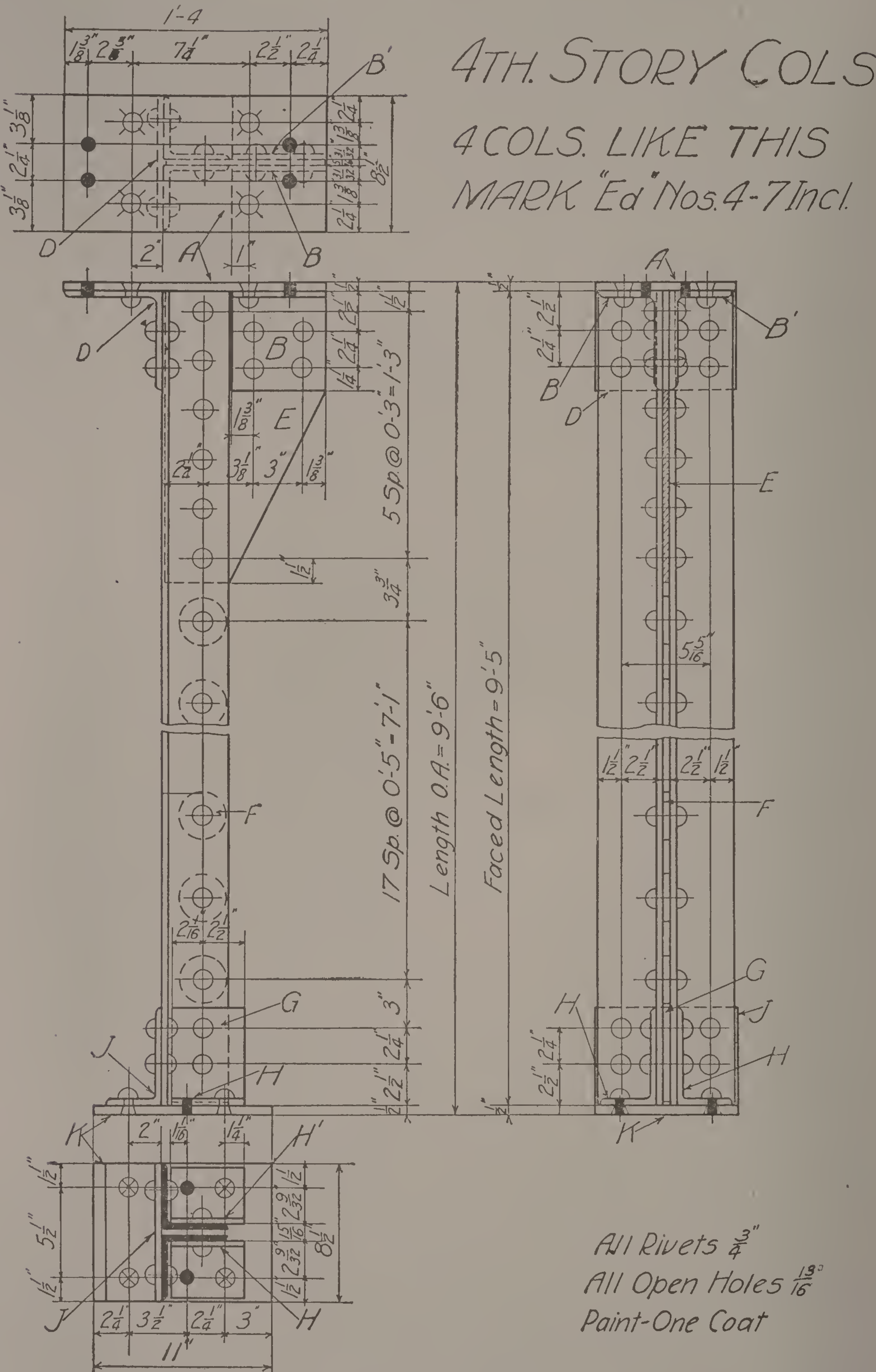

4TH. STORY COLS
4 COLS. LIKE THIS
MARK "Ed" Nos. 4-7 Incl.
5 Sp. @ 0'-3" = 1'-3"
17 Sp. @ 0'-5" = 7'-1"
Length O.A. = 9'-6"
Faced Length = 9'-5"
All Rivets 3/4"
All Open Holes 13/16"
Paint-One Coat

Fig. 212.

Note that the spacing for holes in the cap and in the shelf angles is given in a separate plan, and in this plan the holes are located with respect to each angle and are also located by measurement between holes on opposite sides of the axis of the column. This is advisable in case there is any variation in the measurement back to back of the column angles, or between outside faces of angles. After the column is riveted up, other measurements can be adjusted to the over-all measurements between holes; this measurement is also useful in checking.

The cap angles are bolted on for shipment. In many cases it would be impossible to place a beam between the webs of two columns without taking off the cap angles; for this reason, cap angles on the web should always be shipped bolted on. Cap angles on the flange, in many cases, do not require to be bolted on. Where there are flange plates, the rivets must either be flattened or the beam cut short to allow clearance for the rivet heads. Where the spacing between the vertical lines of rivets in the flange is sufficient to allow the flange of the beam to be lowered between them, the cap angles could be bolted on and the rivets would not then need to be flattened.

The draftsman should constantly have clearly in mind what is necessary to enable the structure to be erected. The details must often be modified in some way to avoid a construction in which it is impossible to erect some member.

The outstanding legs of shear angles under the brackets are here shown riveted together. As previously stated, many details are made with only one shear angle, and where two are used they are not always riveted together. For ordinary loads it is not essential to rivet them together, but is better construction and should always be done where the loads are very considerable.

Fig. 212 shows the detail of a column composed of two angles, back to back, and Fig. 213 gives the bill of material. The only loads in this case are from the beams over the top. If a beam was framed into the shaft of the column parallel with the axis of the two adjacent legs, a connection of a plate riveted to these angles with shelf angles riveted to this plate similar to what is shown for the head, could have been used. The student should study carefully the bill of material of this column, and thoroughly understand each item and the notes regarding the shop work to be done.

Fig. 214 shows the second section of a box column made of two channels with flange plates. Table V, Part I, gives the distance back to back of channels, in order that the radius of gyration shall be equal about each axis. In a box column the distance back to back of channels, should never be less than this. The Carnegie Company and most other shops have standard spacings for such columns which should in general be followed.

As the flange plates on this section are not as thick as those on

Bill of Material for 4 Columns

ITEM	NO. of PIECES	KIND	SIZE	LENGTH FEET	LENGTH INCHES	WORK
	8	ANGLES	$4'' \times 4'' \times \frac{5}{16}''$	9	5	FACED BOTH ENDS
A	4	PLATES	$8\frac{1}{2}'' \times \frac{1}{2}''$	1	4	COUNTERSUNK
B-B'	8	ANGLES	$6'' \times 4'' \times \frac{3}{8}''$	0	$5\frac{3}{4}$	
D	4	"	$6'' \times 6'' \times \frac{3}{8}''$	0	$8\frac{1}{2}$	
E	4	PLATES	$9\frac{3}{4}'' \times \frac{5}{16}''$	1	6	BEVELLED - FACED
F	72	FILLERS	$2\frac{3}{4}'' \times \frac{5}{16}''$			
G	4	"	$3\frac{3}{4}'' \times \frac{5}{16}''$	0	$5\frac{3}{4}$	
H-H'	8	ANGLES	$6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	0	$4\frac{9}{16}$	
J	4	"	$6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	0	$8\frac{1}{2}$	
K	4	PLATES	$8\frac{1}{2}'' \times \frac{1}{2}''$	0	11	COUNTERSUNK

Fig. 213.

Bill of Material for 3 Columns

ITEM	NO. of PC'S.	KIND	SIZE	WT. per FOOT	LENGTH FEET	LENGTH INCHES	WORK
	6	CHANNELS	$10''$	$25\#$	22	$11\frac{1}{2}$	FACED BOTH ENDS
	6	PLATES	$12'' \times \frac{3}{4}''$		22	$11\frac{1}{2}$	" " "
A	3	"	$12'' \times \frac{1}{2}''$		0	$11\frac{1}{2}$	COUNTERSUNK
B	6	"	$12'' \times \frac{1}{2}$		1	$6\frac{1}{2}$	
C	12	ANGLES	$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}$		0	8	
D-D'	12	"	$4'' \times 3'' \times \frac{3}{8}$		1	1	BEVELLED
E-E'	12	"	$6'' \times 4'' \times \frac{1}{2}''$		1	1	"
F	12	"	$3'' \times 3'' \times \frac{5}{16}''$		1	$2\frac{1}{2}$	" FITTED
G	12	"	$3'' \times 3'' \times \frac{5}{16}''$		0	$8\frac{1}{2}$	" "
H	12	FILLERS	$3'' \times \frac{1}{2}''$		0	$8\frac{3}{4}''$	
J	12	"	$2\frac{3}{4}'' \times \frac{1}{2}''$		0	3	
K	12	ANGLES	$6'' \times 4'' \times \frac{1}{2}''$		0	8	
L	6	FILLERS	$8\frac{3}{4}'' \times \frac{1}{4}''$		1	0	SHIP BOLTED

Fig. 215.

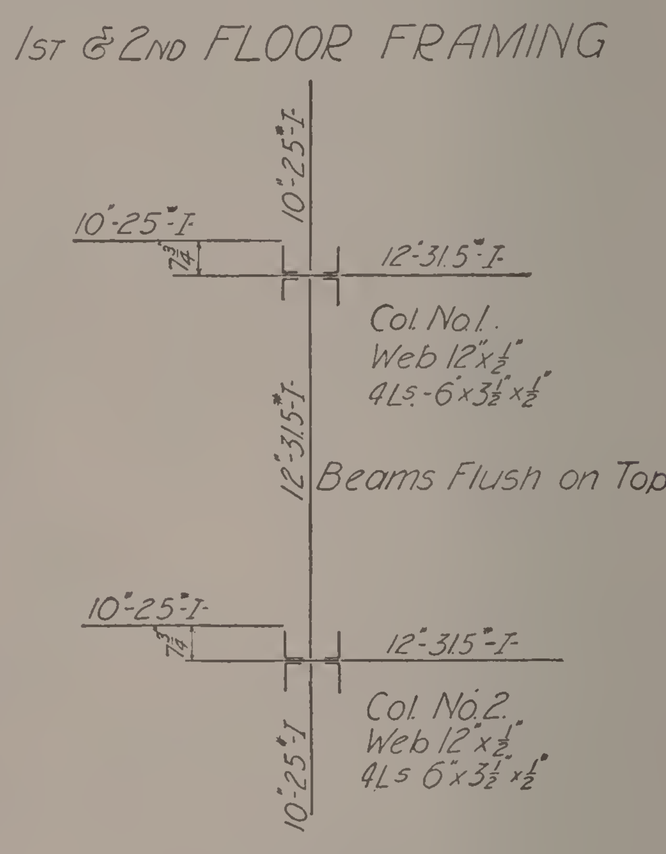


Fig. 216.

the lower section, it is necessary to ship filler plates bolted to the column.

There are two beams framed to each flange of this column so that the shear angles are spread to come as nearly as practicable under the web of the beams. These angles cannot always be made to come directly under the web on account of the relation between the spacing of beams and the spacing of rivets through flanges of

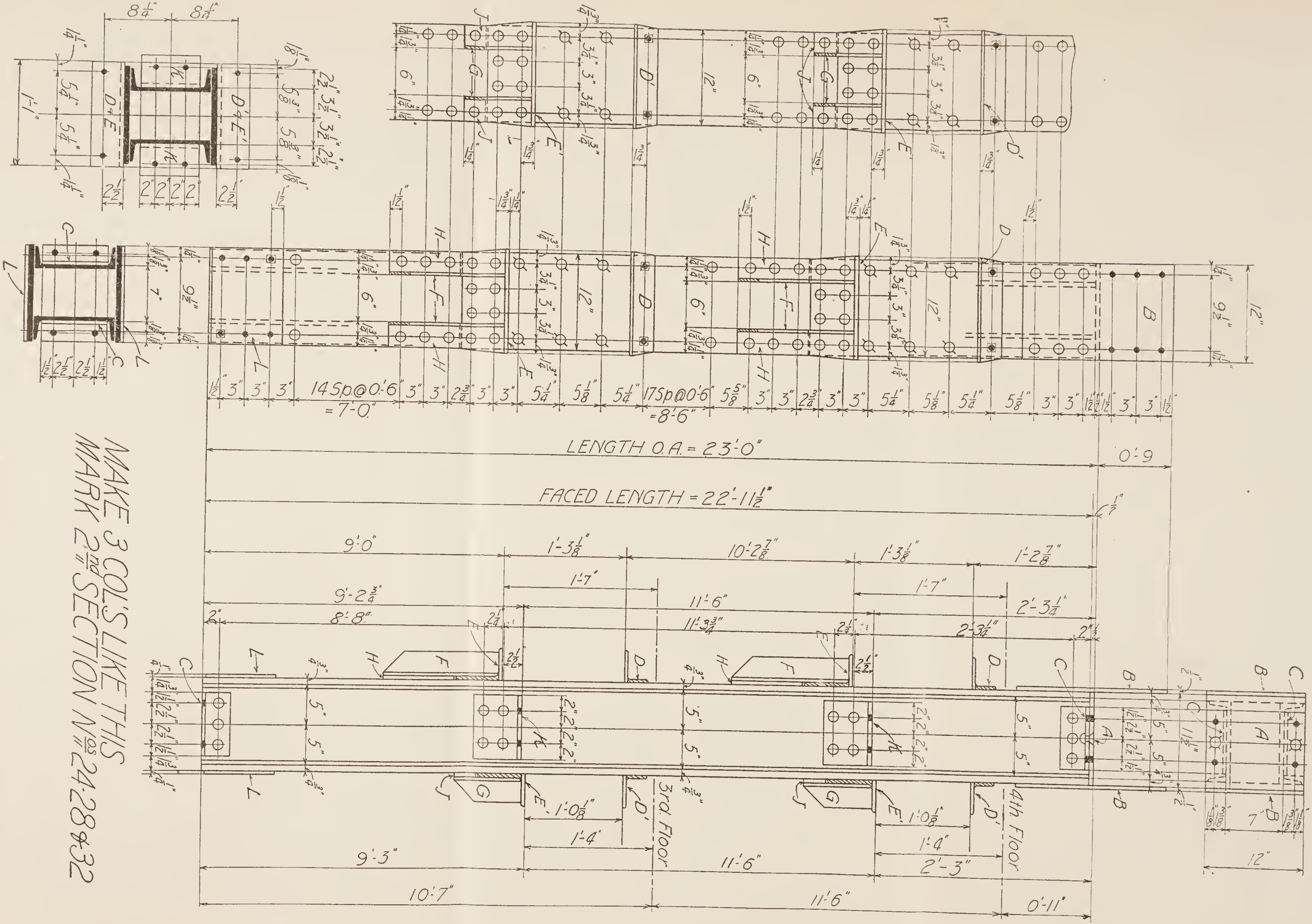

MAKE 3 COLS. LIKE THIS
MARK 2nd SECTION Nos 24-28 & 32
LENGTH O.A. = 23'-0"
FACED LENGTH = 22'-11½"
14 Sp @ 0'-6" = 7'-0"
17 Sp @ 0'-6" = 8'-6"
3rd. Floor
4th Floor

FIG. 214.

channels of columns. Some variation in size of angles can be made, however, at times to effect this result.

Where box columns are used, it is better to keep the spacing back to back of channel the same throughout all sections. If this is less in the upper sections, it brings the load of this section on to the horizontal splice plate between the sections. The distance between the cap and shelf angles is generally $\frac{1}{8}$ in. more than the depth of the beam, to allow for clearance. The rivets between the cap and shelf angles are flattened here, as with one beam in position there would not be space to lower the other beam between the rivet heads.

Fig. 215 gives the bill of material for these box columns. Fig. 216 shows the framing of the beams coming on columns No. 1 and No. 2. detailed in Fig. 217. This column has a heavy steel base riveted to it. The load on the section is 265,000 pounds and it will be seen therefore that the rivets in the shear plates are amply sufficient for the portion of the load coming upon them. The plate W riveted to the web increases the bearing area of the foot of the column and adds somewhat to the efficiency of the base.

In this connection and in such cases where shear angles are used over a shelf angle involving the use of a filler, below the shelf angle and back of the shear angles, as shown by the details of this column, the student should note the difference between a tight and a loose filler.

Fillers G and R are loose fillers. They have no rivets holding them individually to the main members. The stress in the rivets through such a filler does not go into the filler, as there are no extra rivets to take it out again from the filler to the main members. Such rivets, therefore, are subject to bending if calculated for their full value. They should not be considered for more than one-half the value of rivets directly connecting the main members. Filler W is a tight filler as regards the two rivets through the angles X on the axis of the column. A tight filler has provision by additional rivets for taking the same amount of stress from itself to the main member as it receives.

The open holes shown in the base plate are for anchoring to the footing—such heavy columns are not usually anchored except in special cases; it is well, however, to provide for this if there is any possibility of its being required.

Bill of Material for 2 Columns

ITEM	NO. of PIECES	KIND	SIZE	LENGTH		WORK
				FEET	INCHES	
	2	WEB PLS.	$12'' \times \frac{1}{2}''$	24	$8\frac{3}{4}$	FACED BOTH ENDS
	8	FLG. Ls	$6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	24	$8\frac{3}{4}$	" " "
A	4	SPLICE PLS	$12\frac{1}{2}'' \times \frac{1}{2}''$	1	$6\frac{1}{2}$	BEVELLED
B	4	ANGLES	$6'' \times 4'' \times \frac{1}{2}''$	0	$10\frac{3}{4}$	SHIP BOLTED
C	4	FILLERS	$5'' \times \frac{1}{2}''$	0	$5\frac{3}{4}$	" "
D	4	ANGLES	$6'' \times 6'' \times \frac{3}{8}''$	0	$8\frac{7}{8}$	" " BEVELLED
E	4	"	$6'' \times 6'' \times \frac{1}{2}''$	0	$8\frac{7}{8}$	BEVELLED
F	4	STIFF. Ls	$4'' \times 3'' \times \frac{3}{8}''$	0	$8\frac{1}{2}$	" FITTED
G	4	FILLERS	$2\frac{3}{8}'' \times \frac{1}{2}''$	0	$2\frac{3}{4}$	
H	4	ANGLES	$6'' \times 6'' \times \frac{3}{8}''$	0	5	SHIP BOLTED
J	4	"	$6'' \times 6'' \times \frac{1}{2}''$	0	5	
K	8	STIFF. Ls	$3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$	0	$8\frac{1}{2}$	FITTED BEVELLED
L	4	FILLERS	$2\frac{3}{4}'' \times \frac{1}{2}''$	0	5	
M	4	ANGLES	$6'' \times 6'' \times \frac{3}{8}''$	0	$10\frac{3}{4}$	SHIP BOLTED
M	4	FILLERS	$3'' \times \frac{1}{2}''$	0	5	" "
N	4	ANGLES	$6'' \times 6'' \times \frac{1}{2}''$	0	$10\frac{3}{4}$	
O	4	"	$6'' \times 6'' \times \frac{3}{8}''$	0	8	SHIP BOLTED
P	4	"	$6'' \times 6'' \times \frac{1}{2}''$	0	8	
Q	8	STIFF. Ls	$4'' \times 3 \times \frac{5}{16}''$	0	$8\frac{1}{2}$	FITTED BEVELLED
R	4	FILLERS	$2\frac{3}{4}'' \times \frac{1}{2}''$	0	8	
S	4	PLATES	$18'' \times \frac{1}{2}''$	2	0	FACED BEVELLED
T	4	ANGLES	$6'' \times 4'' \times \frac{1}{2}''$	2	0	BEVELLED
U	8	STIFF. Ls	$4'' \times 3'' \times \frac{3}{8}''$	1	$5\frac{1}{2}$	FITTED BEVELLED
V	4	FILLERS	$8'' \times \frac{1}{2}''$	0	$11\frac{3}{4}$	
W	4	PLATES	$10\frac{3}{4}'' \times \frac{1}{2}''$	1	0	FACED
X	4	ANGLES	$6'' \times 4'' \times \frac{1}{2}''$	0	$10\frac{3}{4}$	
Y	4	FILLERS	$5'' \times \frac{1}{2}''$	0	$11\frac{1}{2}$	
Z	2	BASE PLS.	$24'' \times \frac{3}{4}''$	2	0	COUNTERSUNK
	6	BOLTS	$\frac{3}{4}''$	0	$3\frac{1}{2}$	
	12	"	$\frac{3}{4}''$	0	3	
	20	"	$\frac{3}{4}''$	0	2	

Fig. 218.

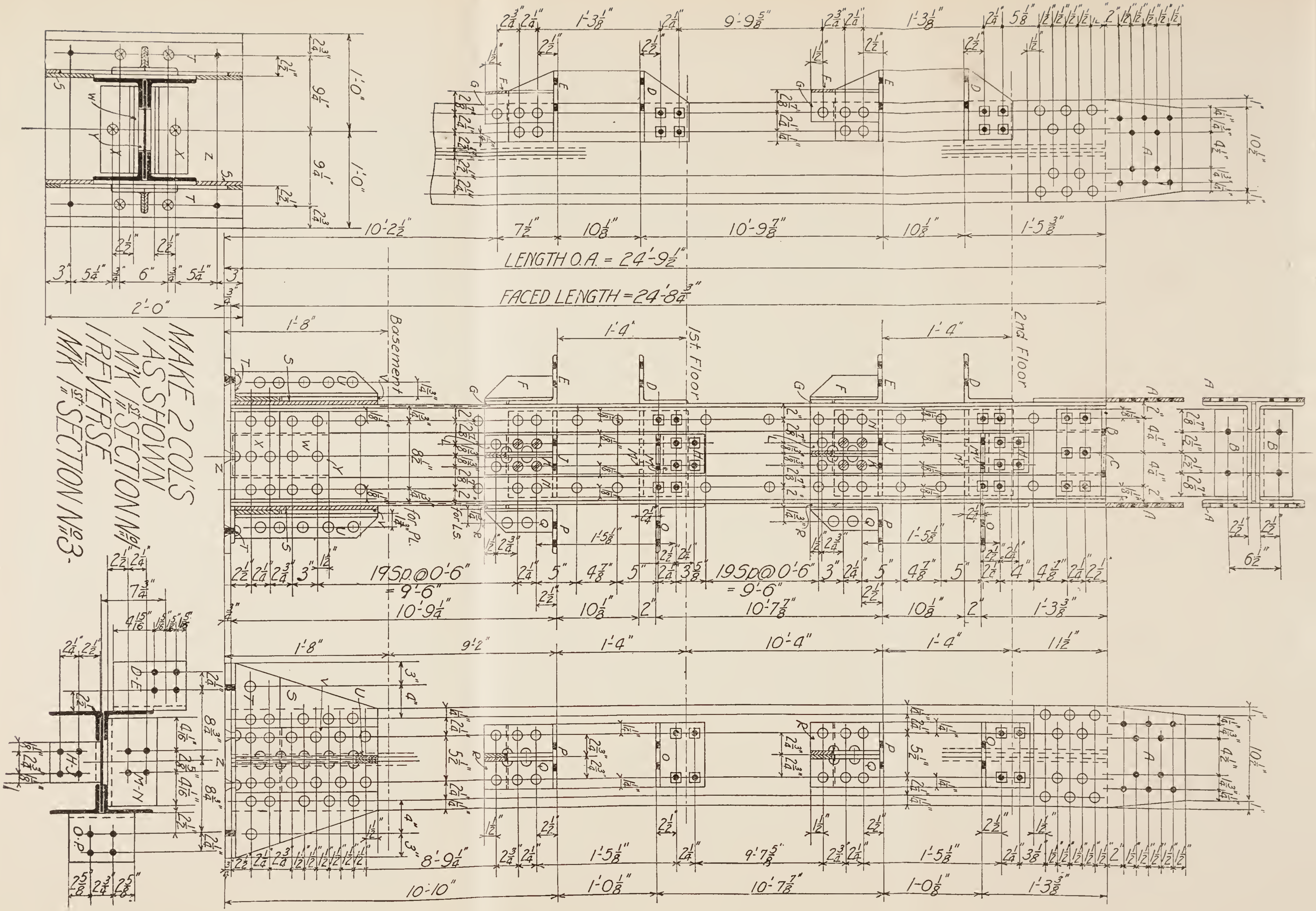

LENGTH O.A. = 24'-9½"
FACED LENGTH = 24'-8¾"
MAKE 2 COL'S
1 AS SHOWN
MK 1st SECTION No 1
1 REVERSE
MK 1st SECTION No 3
Basement
1st Floor
2nd Floor
19 Sp. @ 0'-6" = 9'-6"

FIG. 217.

In the connection for floor beams it will be noted that a 10-in. beam comes in on one side of the web and a 12-in. beam on the other side. Such cases often result in special cap angle details in order to provide for riveting without interference on either side. In this case it is impossible to get the upper holes in cap H more than $\frac{3}{4}$ in. from the upper edge of cap M unless these holes are brought nearer the upper edge of H than $1\frac{1}{4}$ in., which it is undesirable to do. It is necessary, therefore, to add an extra filler B B to fill out flush with the angle N so as to be able to rivet. The student should follow the detail through and see just why this condition results from the measurements given.

The four rivets in angles L are countersunk on the far side so as to avoid a filler and riveting through angle N.

The 10-in. beam connecting on the flange of the column at one side of the axis, requires a connection similar to that shown. If the load coming off the axis was very heavy, a deeper shear plate would be used back of the shelf angle, and it would be better to run this shear plate across both angles of the flange, both to provide for the bending on the rivets and also to distribute the load more uniformly with respect to the axis of the column.

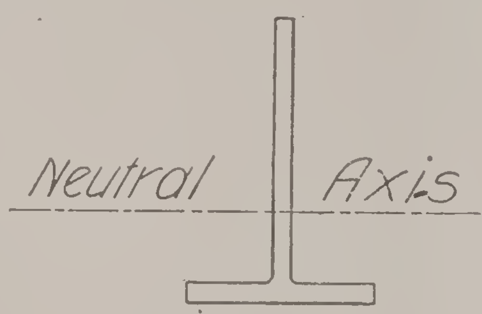


Fig. 222.

There are no standard details for eccentric and special framings. The draftsman must use his judgment and endeavor to get as simple and effective connections as possible.

The section which comes on top of this one has 5-in. angles, in order to use standard spacing in these angles, therefore, the spacing in splice plates has to be on a special gauge and this place is beveled to give a neater appearance when the two sections are riveted together. Fig. 218 gives the bill of material.

Fig. 219 gives the detail of an angle over an opening resting in a wall at one end and framing to a beam at the other. The holes in the horizontal leg are for securing the frame of the window. As this angle has framing on one end only it is not reversible and therefore for the wall on opposite side of the building the angle must be made "opposite hand" or reversed.

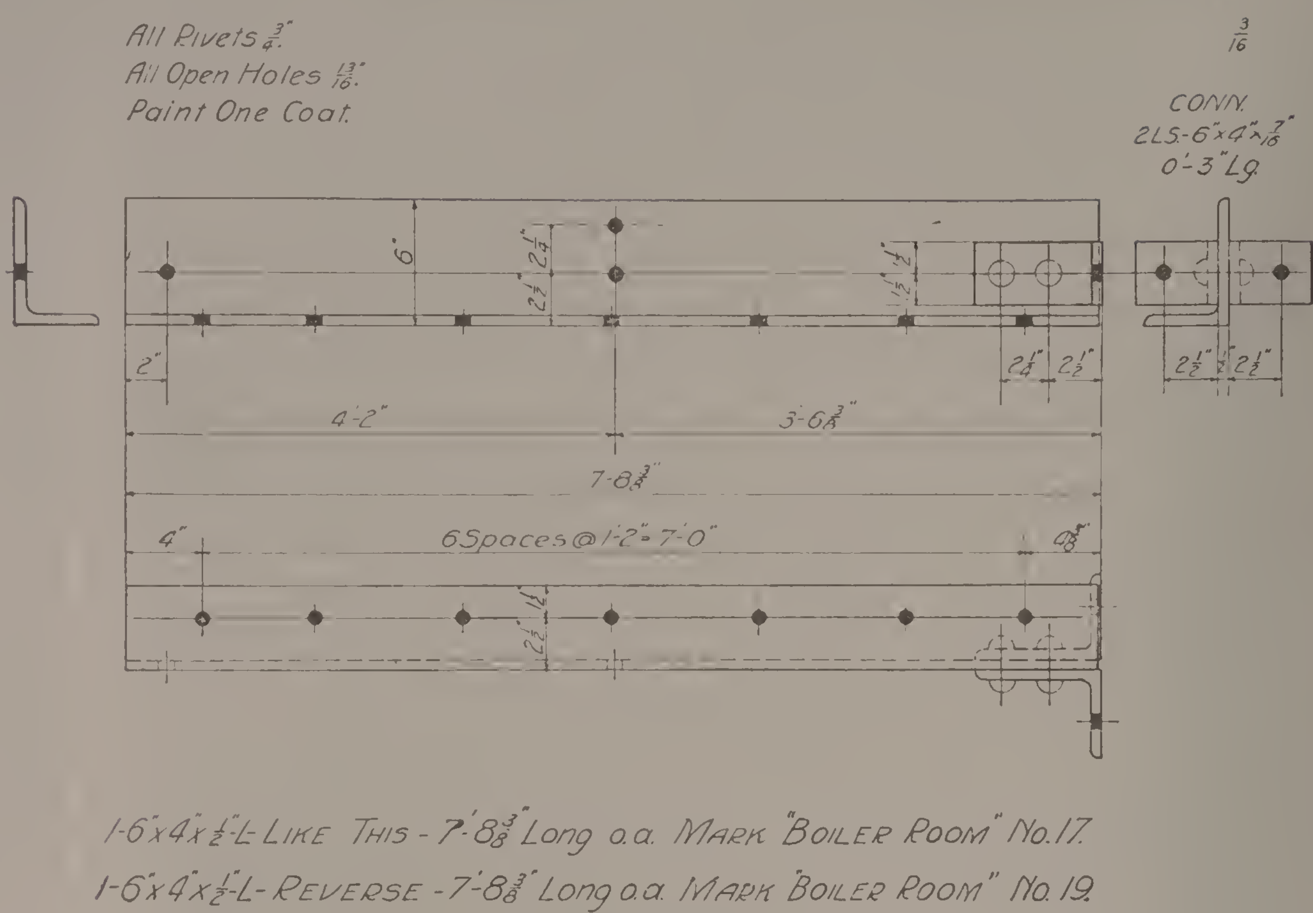


Fig. 219.

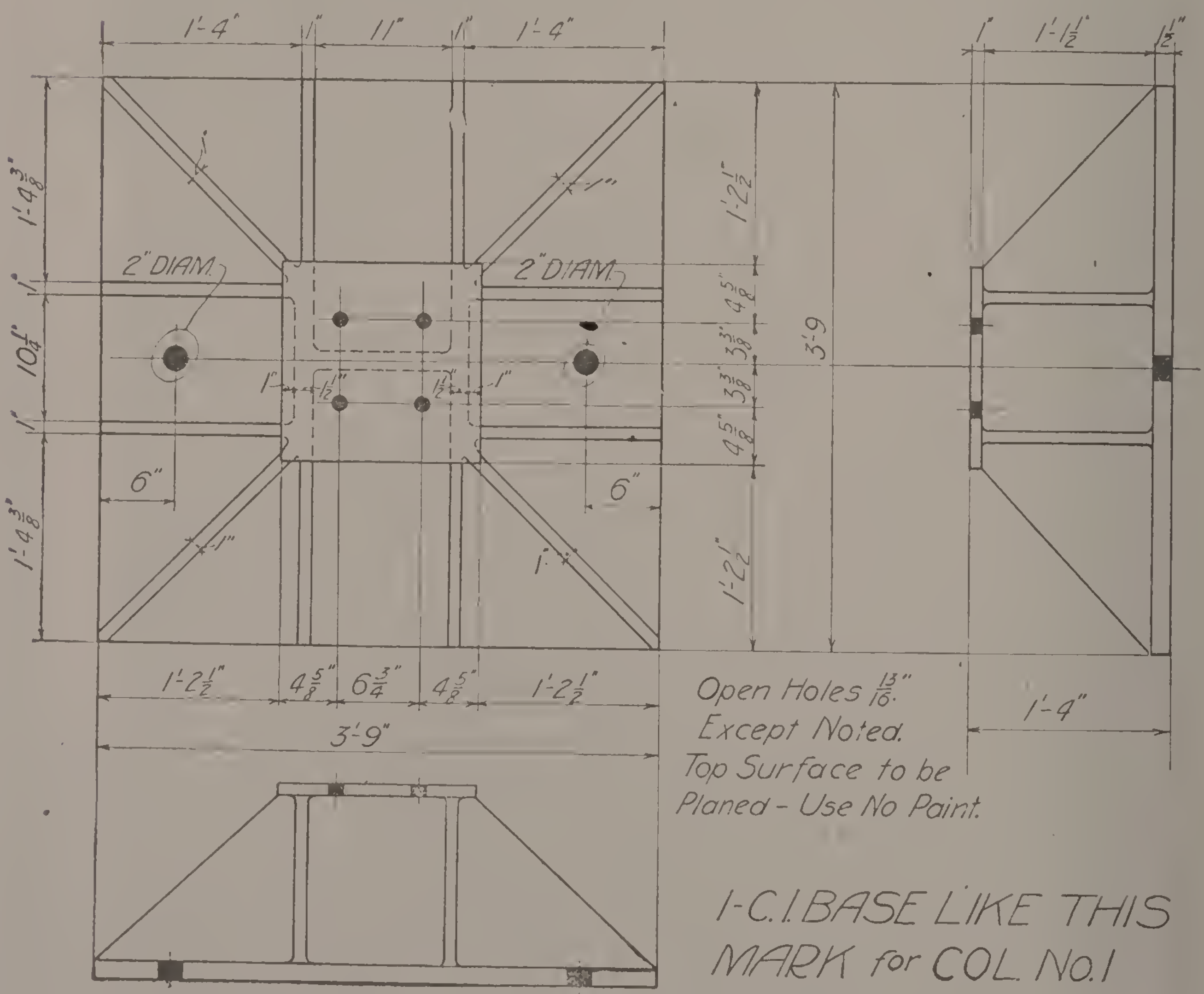
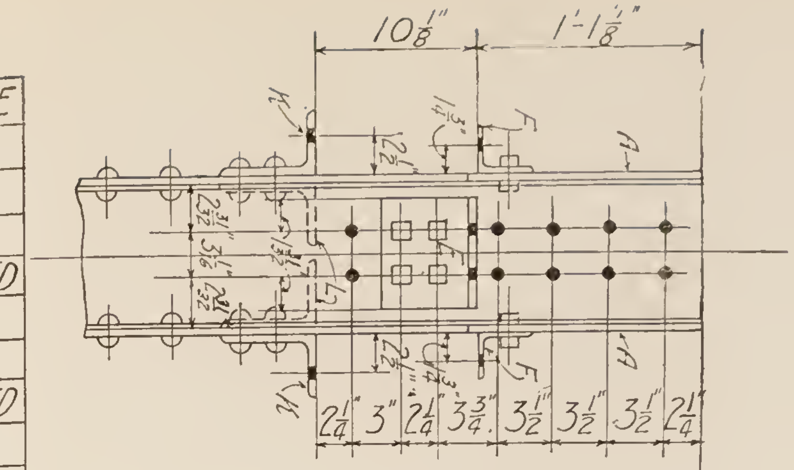
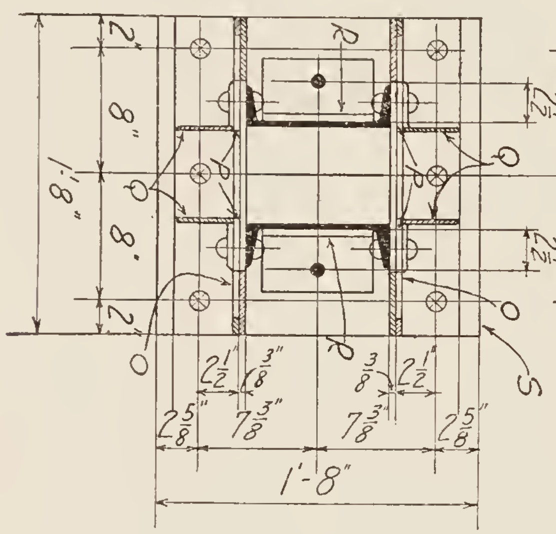


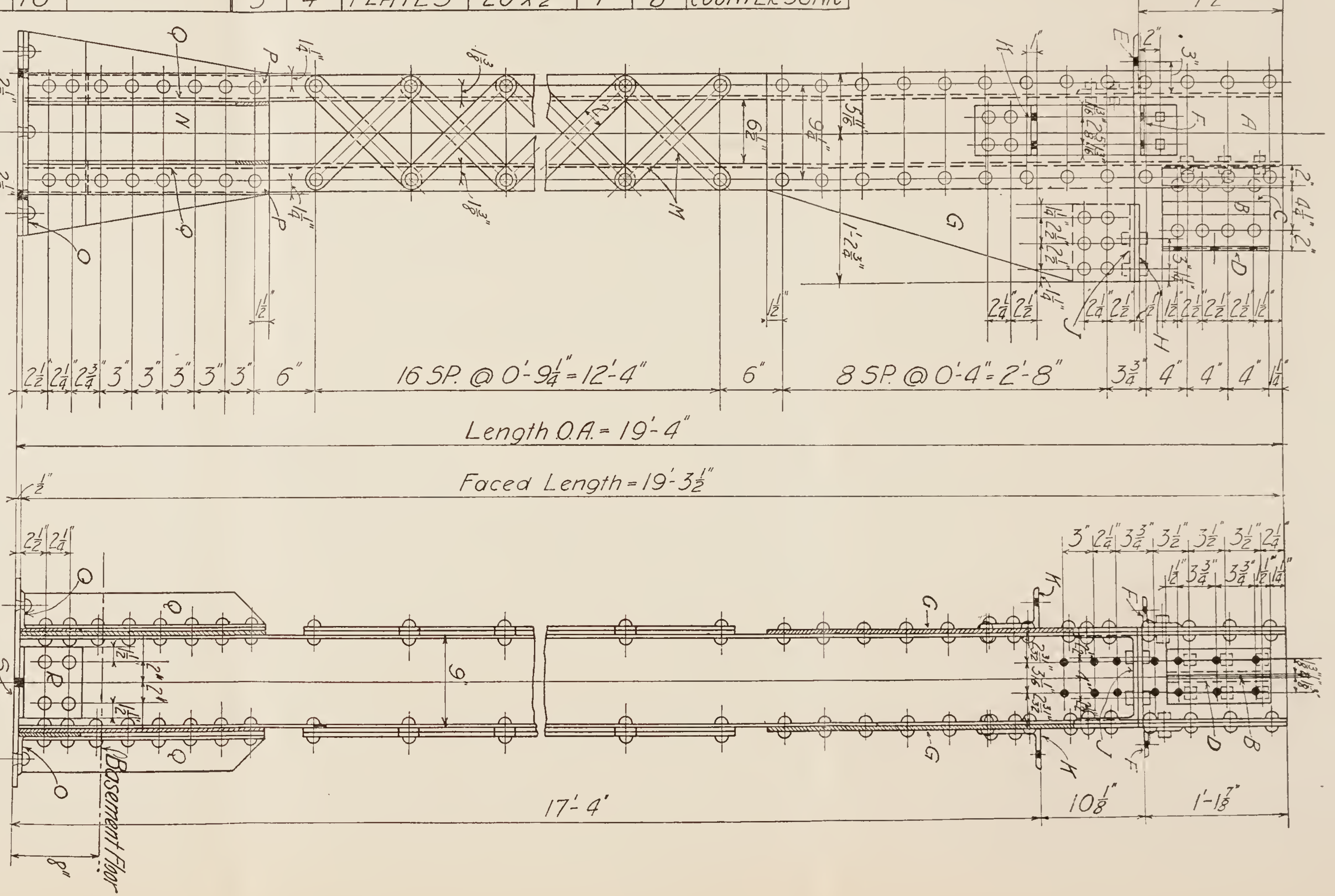
Fig. 221.

Bill of Material for 4 Columns / Bill Continued

ITEM	NO. OF PIECES	KIND	SIZE	LENGTH FEET	LENGTH INCHES	WORK
	8	CHANNELS	9"-13.25#	19	$3\frac{1}{2}$	FACE BOTH ENDS
A	8	PLATES	$11\frac{3}{8}$" x $\frac{1}{2}$"	1	$2\frac{1}{2}$	" " "
B	4	"	8" x $\frac{1}{4}$"	0	$10\frac{1}{2}$	
C	8	ANGLES	$3\frac{1}{2}$" x $2\frac{1}{2}$ x $\frac{5}{16}$"	0	$10\frac{1}{2}$	SHIP BOLTED
D	8	"	"	0	$10\frac{1}{2}$	
E	4	"	6" x 6" x $\frac{1}{2}$"	0	7	
F	8	"	$3\frac{1}{2}$" x 3" x $\frac{5}{16}$"	0	5	
G	8	PLATES	$20\frac{7}{16}$" x $\frac{1}{2}$"	3	0	FACE ONE END BEVELLED
H	4	"	$7\frac{1}{2}$" x $\frac{1}{2}$"	0	10	
J	4	PLATES	$7\frac{1}{2}$" x $\frac{1}{2}$"	1	9	BENT TO SHAPE
K	8	ANGLES	6" x 4" x $\frac{1}{2}$"	0	5	
L	8	"	"	0	5	
M	64	BARS	2" x $\frac{3}{8}$"	1	$3\frac{1}{2}$	ENDS ROUNDED
N	8	PLATES	20" x $\frac{3}{8}$"	2	0	BEVELLED FACED
O	8	ANGLES	6" x 4" x $\frac{1}{2}$"	1	8	BEVELLED
P	16	FILLERS	3" x $\frac{1}{2}$"	1	6	
Q	16	ANGLES	$3\frac{1}{2}$" x 3 x $\frac{3}{8}$"	1	$11\frac{1}{2}$	BEVELLED FITTED
R	8	"	6" x 4" x $\frac{1}{2}$"	0	7	
S	4	PLATES	20" x $\frac{1}{2}$"	1	8	COUNTERSUNK







4 COLS LIKE THIS
MARK "E" LOWER SECTION
NOS 1-4 INCL.

FIG. 223.

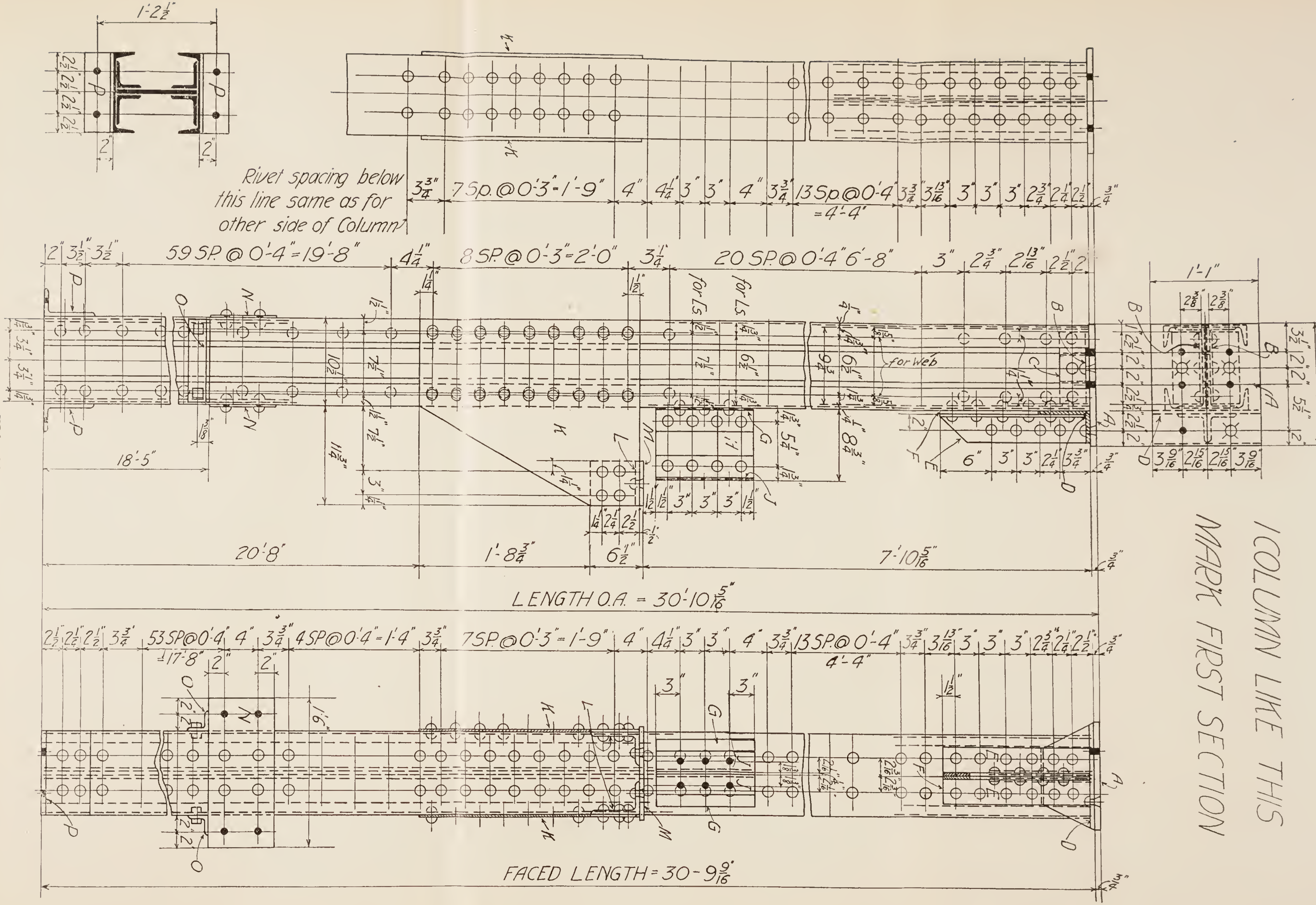

1'-2½"
Rivet spacing below this line same as for other side of Column
7 Sp. @ 0'-3" = 1'-9"
13 Sp. @ 0'-4" = 4'-4"
59 SP. @ 0'-4" = 19'-8"
8 SP. @ 0'-3" = 2'-0"
20 SP. @ 0'-4" 6'-8"
for Ls.
for Web
1'-1"
1'-3"
18'-5"
20'-8"
1'-8¾"
6½"
7'-10 5/16"
LENGTH O.A. = 30'-10 5/16"
53 SP @ 0'-4" = 17'-8"
4 SP @ 0'-4" = 1'-4"
7 SP. @ 0'-3" = 1'-9"
13 SP. @ 0'-4" 4'-4"
FACED LENGTH = 30'-9 9/16"
I COLUMN LIKE THIS
MARK FIRST SECTION

FIG. 224.

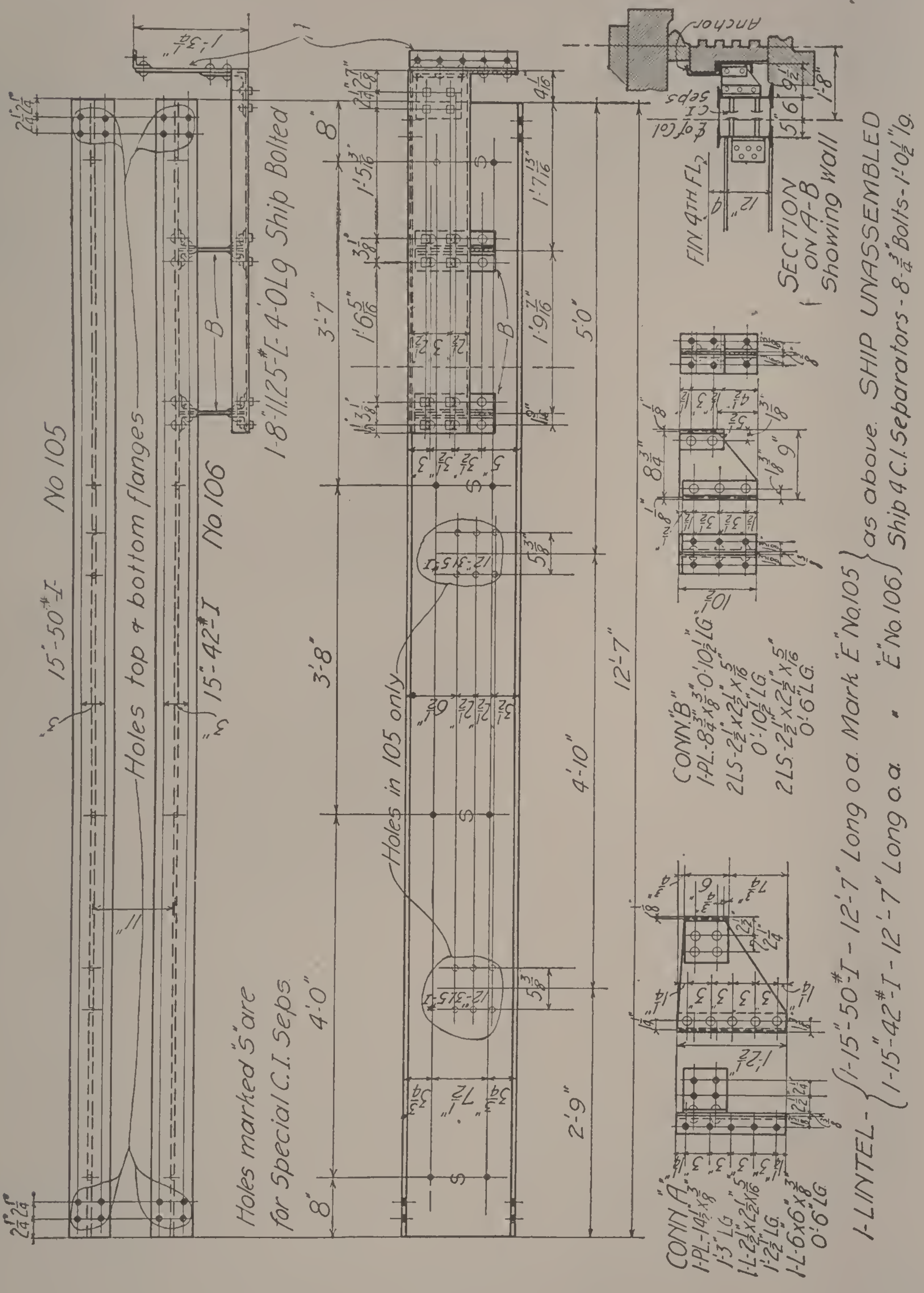

No 105
15"-50# I
Holes top & bottom flanges
No 106
15"-42# I
1-8"-11.25# [- 4'-0 Lg Ship Bolted
Holes marked "S" are for Special C.I. Seps
Holes in 105 only
SECTION ON A-B Showing wall
Anchor
CONN "A"
CONN "B"
1-LINTEL-
1-15"-50# I - 12'-7" Long o.a. Mark "E" No.105
1-15"-42# I - 12'-7" Long o.a. "E" No.106
as above. SHIP UNASSEMBLED
Ship 4 C.I. Separators - 8-3/4" Bolts - 1'-0½" lg.

Fig. 220.

Fig. 220 gives the detail of a spandrel girder and shows in outline the relation of the stone facing to the girder. This wall section is a pier and the width of it is indicated by the length of the 8-in. channel at the right-hand end. At the opposite end the wall is only a covering for the column and is carried on the column. This channel supports the block surrounding it, which in turn supports the mass of stone above; the course below is hung by anchors, to the 8-in. channel. The channel is supported by brackets from the beams which are detailed separately for clearness, although they are shipped riveted to the beam. The connection A runs back to the column. There are two floor beams framed to the girder, but as the space between center of beams is 11 in. there is sufficient room to drive rivets passing through only one beam, and this is preferable, therefore, to using through bolts.

Note the specification "Ship Unassembled". This means that the two beams are not bolted together for shipment.

Fig. 221 gives the detail of a cast iron base for a plate and angle column having a 12-in. web. The outlines of the members of the column shaft should be carried down by similar outlines in the cast iron base. In this case the box of the base is H-shaped and the centers correspond to the centers of the shaft members. The thickness of this box under the column must be sufficient to carry the whole load of the column without exceeding the safe compressive strength of cast iron. The size of the base depends upon the area required to distribute the load on the footing. The purpose of the ribs and base is to resist the tendency to break, due to this uniformly distributed load on the footing. Failure would generally occur through the bending action of the portion of the base projecting beyond the box. The moment on this may be figured as for a beam fixed at one end and free at the other and loaded uniformly with the load per unit of bearing surface.

Taking one rib and the base half way on each side between the next rib would give a section at the box, which may be taken as the fixed end, similar to Fig. 222. Calculate then the position of the neutral axis and figure the moment of inertia of the section about this axis. Having determined the bending moment for the width between the ribs, the fiber stress in tension and compression can be

found by the formulas used in calculation of beams. $f = \frac{My}{I}$ where M is the bending moment in inch pounds, y the distance from the neutral axis to the extreme fiber, and I the amount of inertia.

A section must of course be assumed at the outset and it may be necessary to modify this to come within the requirements. It is necessary also to calculate the stresses at the most unfavorable section, and to see that there is sufficient metal across the corners to prevent cracking diagonally between the foot of the ribs on adjacent sides.

Different sections of columns require, as previously stated, different sections of box under the column, and this would affect the arrangement of the ribs more or less. These ribs in general should be at an angle of 45 or 60 degrees. In some cases lower bases can be used, but these are of course subject to greater bending strains.

PROBLEMS.

1. Given a 12-in., $31\frac{1}{2}$-lb. beam framed to a column at each end, the distance between faces being 12 ft. $2\frac{1}{2}$ in. The beam has two 7-in., 15-lb. I-beams framed on one side and opposite these in each case is a 12-in., $31\frac{1}{2}$-lb. I-beam. The distance from center of connections to the face of the column at each end is 3 ft. $5\frac{3}{4}$ in. Make a shop detail of the 12-in. girder, all beams being flush on top.

2. In the above problem, if the 7-in. beams frame at the other end to a 12-in., $31\frac{1}{2}$-lb. beam along a wall, both being flush on top, and it is 11 ft. center to center of girders, make shop details covering both 7-in. beams.

3. Given a 15-in., 33-lb. channel framed to a column at each end, the distance being 16 ft. $5\frac{1}{4}$ in. between faces, and the channel having a $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$-in. angle on the back side, with the long leg vertical and $1\frac{3}{4}$ in. from the bottom. A 10-in., 25-lb. beam frames flush with bottom of the channel 5 ft. $4\frac{1}{4}$ in. from face of each column. Make detail of above.

Mill Building Columns. Fig. 223 gives the detail of the columns shown in Fig. 186, Part II, and by the plate on the preceding page. This is a latticed channel column. Each flange is double laced, that is, it has two systems of lattice bars. In many cases such columns have only one system across each flange; in such cases the

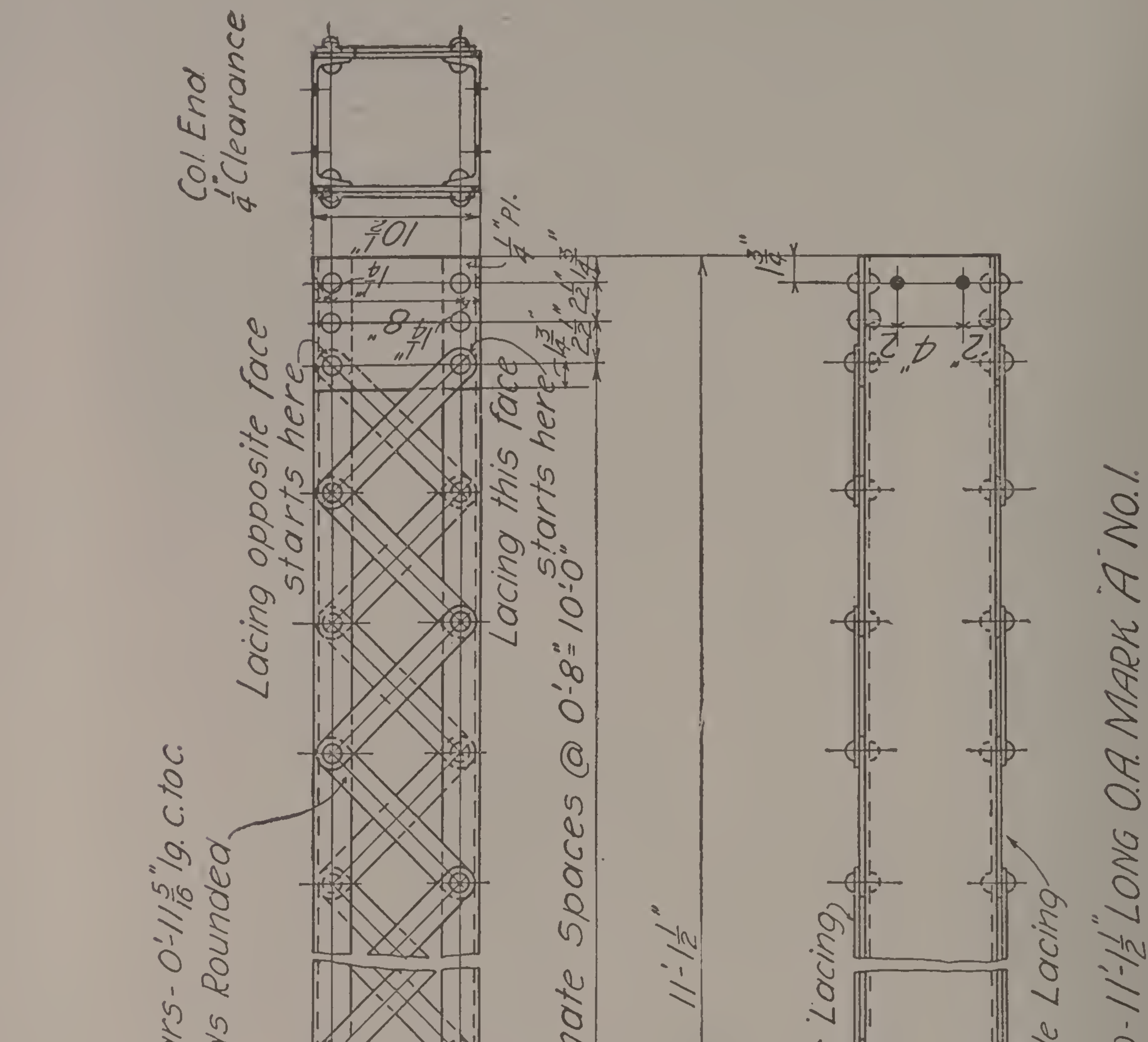

Col. End
1/4" Clearance
Lacing opposite face
starts here
Lacing this face
starts here
60-2"x1/4" Bars-0'-11 5/16" lg. c.to c.
Ends Rounded
15 Alternate Spaces @ 0'-8" = 10'-0"
11'-1 1/2"
Single Lacing
Single Lacing
1-BRACE-2-8"-11.25#-[s-LACED-11'-1 1/2" LONG O.A. MARK 'A' No.1
Rivets 5/8"
Open Holes 13/16"
Paint-One Coat Graphite
Col. End
1/4" Clearance

Fig. 225.

bars on one flange would cross those on the opposite flange; just as if one system shown by Fig. 186 was on one flange and the other on the other flange.

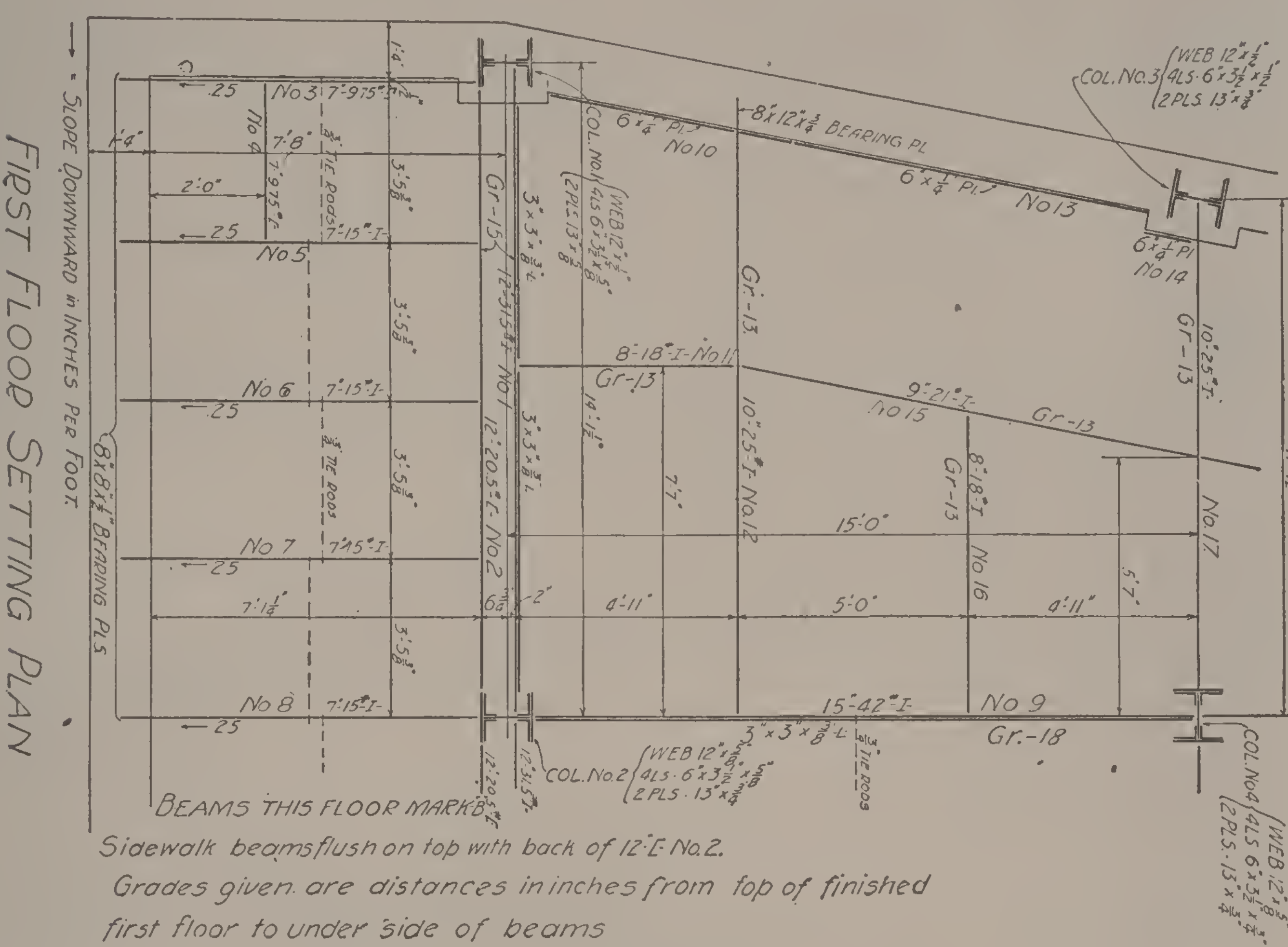


Fig. 226.

This column has a bracket for a crane track girder with a diaphragm bracing the crane girder to the column. The roof column, as shown by Fig. 186, is a plate and angle column and sets down

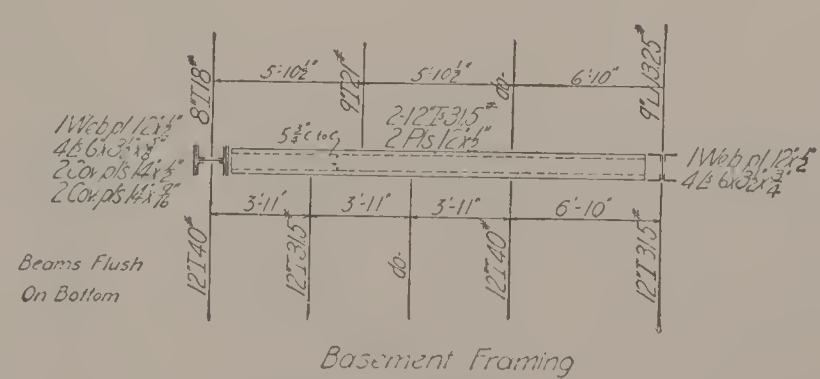


Fig. 227.

between the channels, as the web runs at right angles to the web of the channels. It is always better to avoid re-entrant angles in a plate if possible. In a case like this where a bracket plate comes into the lines of the column at the top and there is a plate the width of the

flanges above this point, it is better to make this a separate plate. If this plate is necessary for the effective area of the column the joint

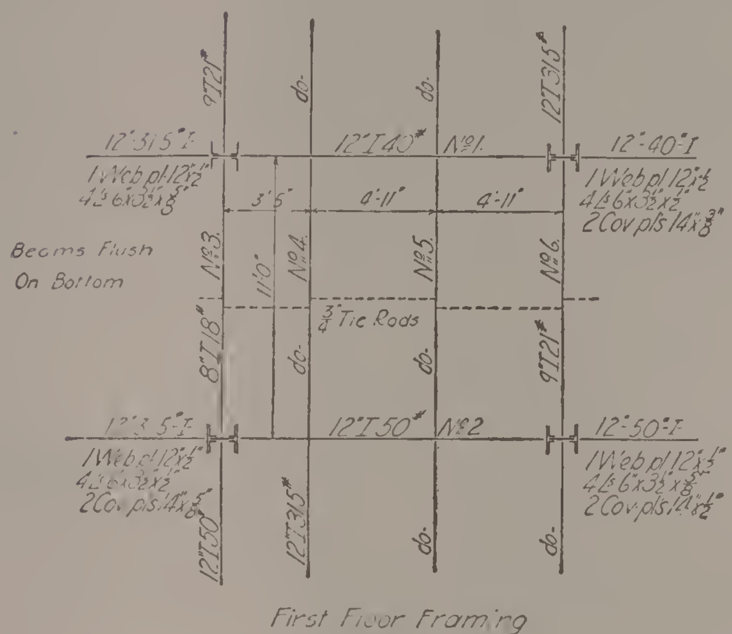
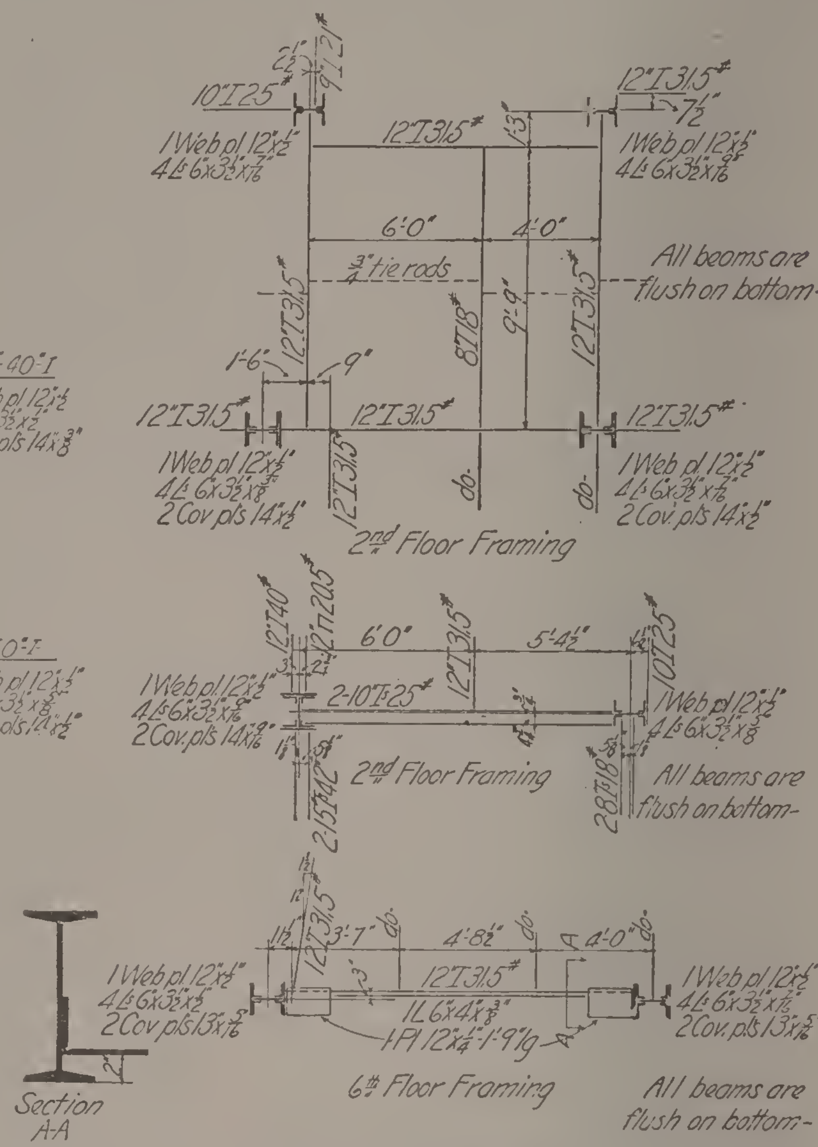


Fig. 228.



Figs. 229, 230, 231.

can be faced. The bracket and shelf angles on the plate are for a beam framed between columns.

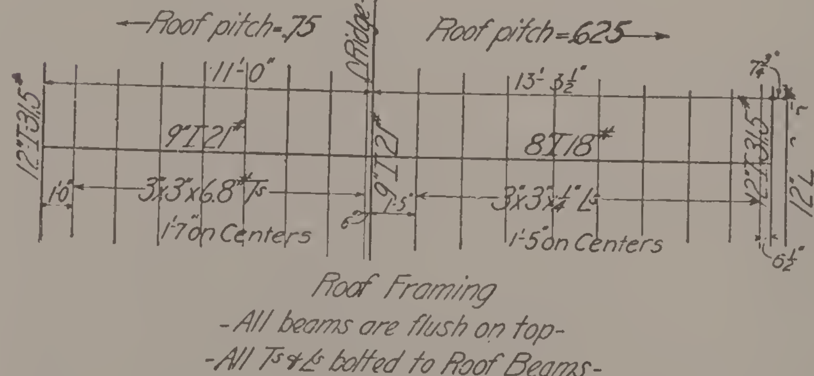
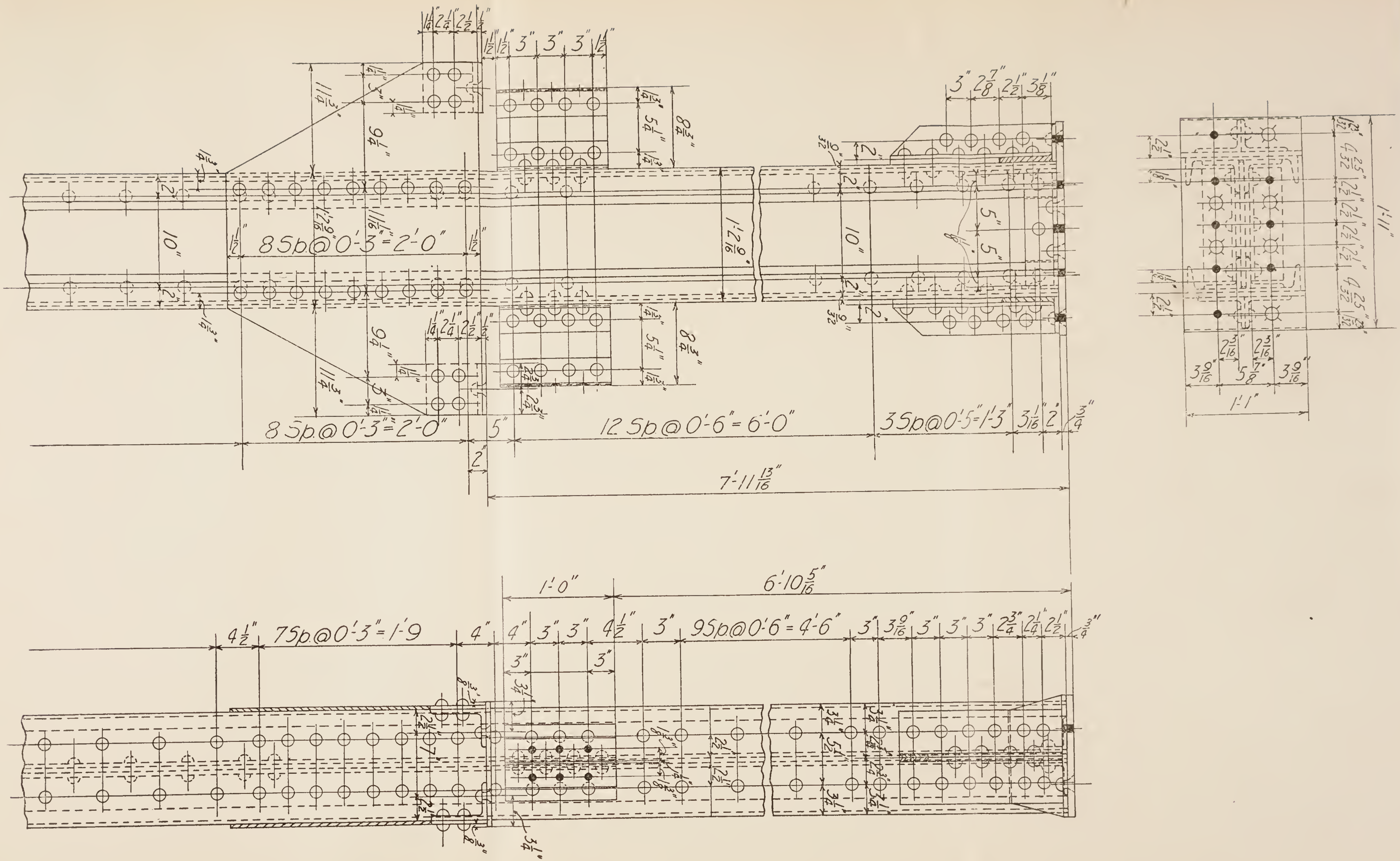


Fig. 232.

The student should be able to follow this detail and understand all the points without further explanation.



8 Sp.@0'-3"=2'-0"
12 Sp.@0'-6"=6'-0"
3 Sp.@0'-5"=1'-3"
7'-11 13/16"
6'-10 5/16"
1'-0"
7 Sp.@0'-3"=1'-9
9 Sp.@0'-6"=4'-6"
1'-1"
1'-11"

FIG. 238.

Fig. 224 shows another type of column made of a web plate and four angles with channels across the flange angles, the flanges being turned in.

There are various reasons for turning the channels with flanges in; here it is desirable to have a 10-in. arch for stiffness, and the thickness of the wall in which this column comes makes it necessary to turn the flanges in; this also allows the column to set flush with the inside face of the wall and gives a smooth surface. Then again, this gives good connection for the cranes girder bracket and for the wind strut below, at N and O.

The top of this column receives a heavy floor girder and another column; the latter column is made of a smaller web so as to provide a seat over the main column members for the girder. Fig. 225 gives a detail of the wind strut which frames between the columns.

In columns of the type shown in Fig. 224, the dimensions must be such as to give room between the flanges of channels, and between the flanges and web, to rivet up the different members.

For light building construction columns are sometimes made of hollow iron pipe fitted with a cast iron cap and base. The dimensions, weights, etc., of standard steam, gas, and water pipe, as manufactured by the American Tube and Iron Co., will be found on page 344, Cambria Handbook. Fig. 233 gives a diagram giving the "strength of wrought iron pipe in compression" according to the formula

$$10750 - 399 \frac{L}{r}$$

in which L = length of column in feet

r = least radius of gyration.

For example, suppose we wish to select a size of pipe suitable for supporting a load of 25,000 pounds, and having a length (or height) of fifteen feet. Along the left hand side of the diagram, under "thousands of pounds" find 25 (*i.e.* 25 thousands), and then find the length (= 15 feet) along bottom line of the diagram. Follow the vertical line at 15 feet until it intersects the horizontal line through 25 thousands, and the nearest inclined line *above* that point will give the diameter of the pipe to be used. In this case a 5-in. diameter will be required.

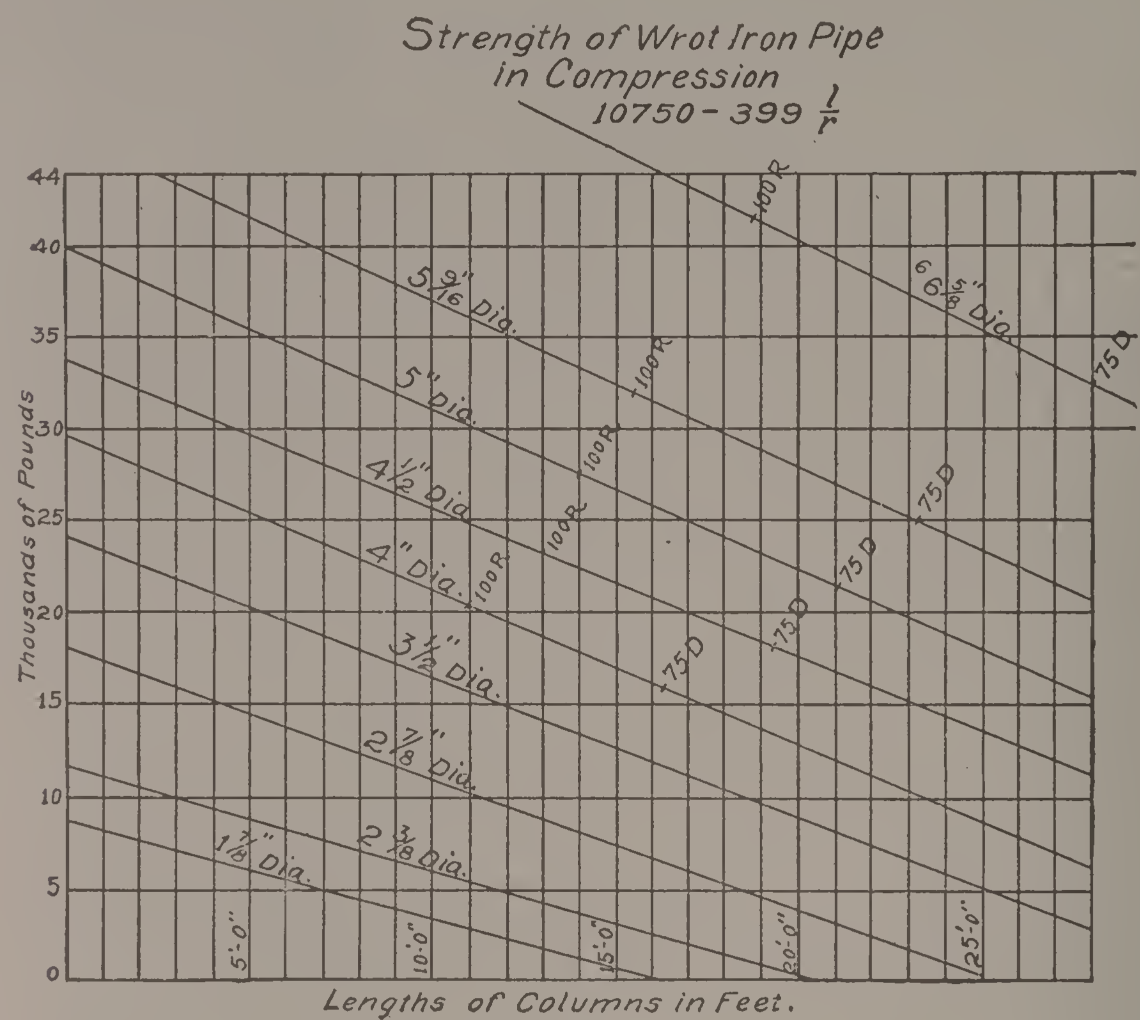


Fig. 234.

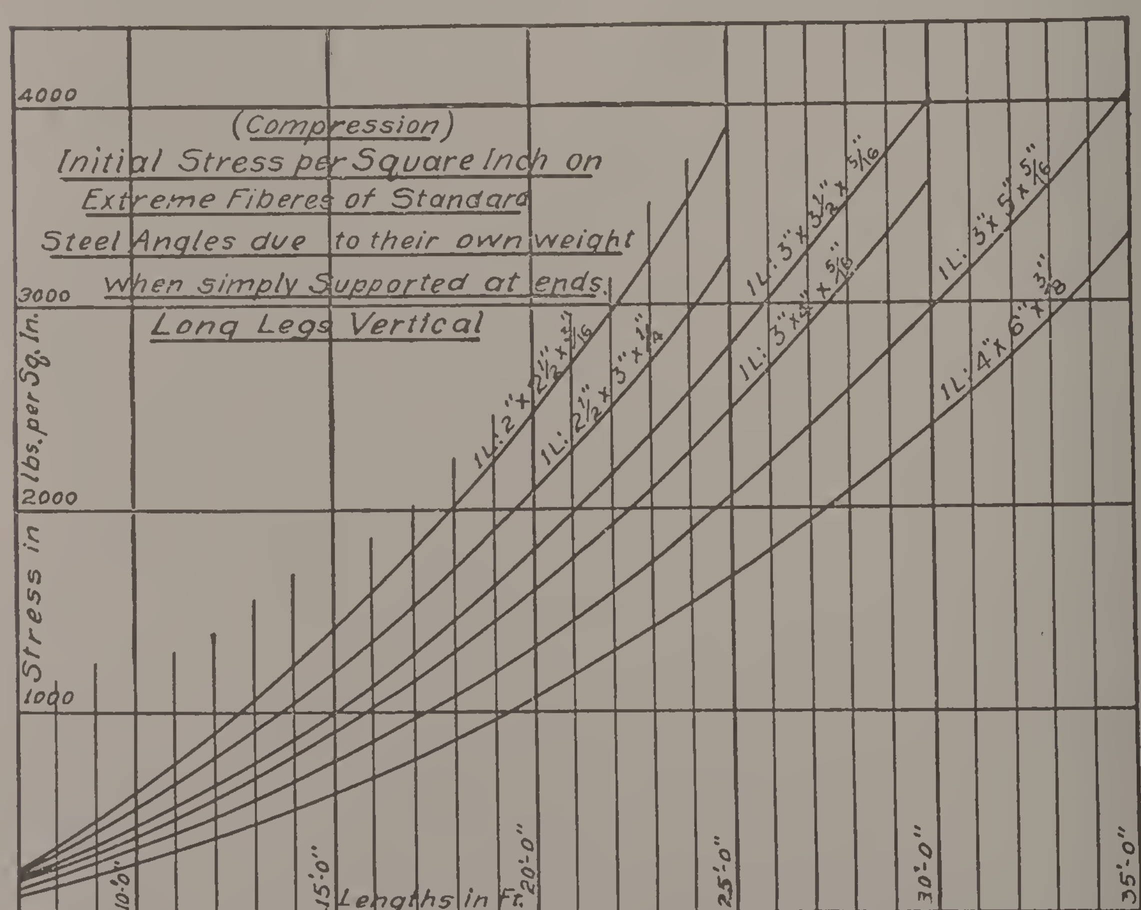


Fig. 235.

Approx. Weight of Steel in Buildings—per Sq. ft. Area covered for use in determining dead load

Length of Building	Span of Roof	Distance between Trusses	Height to Eaves	Pitch of Roof	Load on Roof	Walls to be of	Weight (lb.) per Sq. Foot of— Trusses	Column	Purlins	Framg	Cor. I	Total.	Remarks
49'-3"	In 2 Spans 98'-5	16'-5"	19'-9"	6"	30 lb.	Cor. I	2.38	2.93	2.10	4.22	4.90	16.78	Trav. Crane in half the building 6 T.
112'.6"	66'-0"	18'-9"	26'-3"	6"	30 "		2.07	1.54	2.27	2.27	3.72	11.87	
84'-0"	48'-0"	12'-0"	20'-0"	6"	35 "	Brick curtain walls						16.00	
70'-0"	In 2 Spans 120'-0"	17'-6"	40'-0"	6"	30 "	"	1.57	3.15	1.70	3.43	1.68	14.40	Trav. Crane in half the building 15 Ton.
50'-0"	42'-0"	12'-6"	18'-0"	6"	40 "	Cor. Iron	1.77	1.36	1.93	5.01	5.58	15.65	Ordinary
50'-0"	42'-0	16'-8"	18'-0"	6"	30 "	" "	1.57	1.45	2.83	4.80	5.58	16.33	"
120'-0"	50'-0"	17'-0"	....	6"	55 "	Brick	3.32	....	3.15	0.72	2.50	9.69	N. Y. Blg. Laws
372'-0"	In 2 Spans 115'-0"	19'-6"	21'-0"	4"	....	"	4.81	0.67	2.65	0.67	....	8.80	Plans made by B. and A. R.R.
125'-0"	58'-0"	12'-0"	....	6"	40 "	....	4.52	....	....	0.80	....	5.32	1600 lb. on bot. ch. 6 Js = 8'-0"
196'-9"	One Span 82'-0"	16'-4¾"	12'-7"	4"	30 "	Cor.-I	4.05	0.86	2.08	1.81	3.00	11.80	Ordinary Export
59'-1"	40'-10"	11'-3"	....	6"	Plans Furnished	Brick	3.43	....	3.78	1.59	....	8.80	
56'-0"	29'-0"	11'-2½"	25'-0"	6"	40 lb.	Brick curtain walls	1.84	3.08	2.38	2.45	2.65	12.40	

Fig. 236.

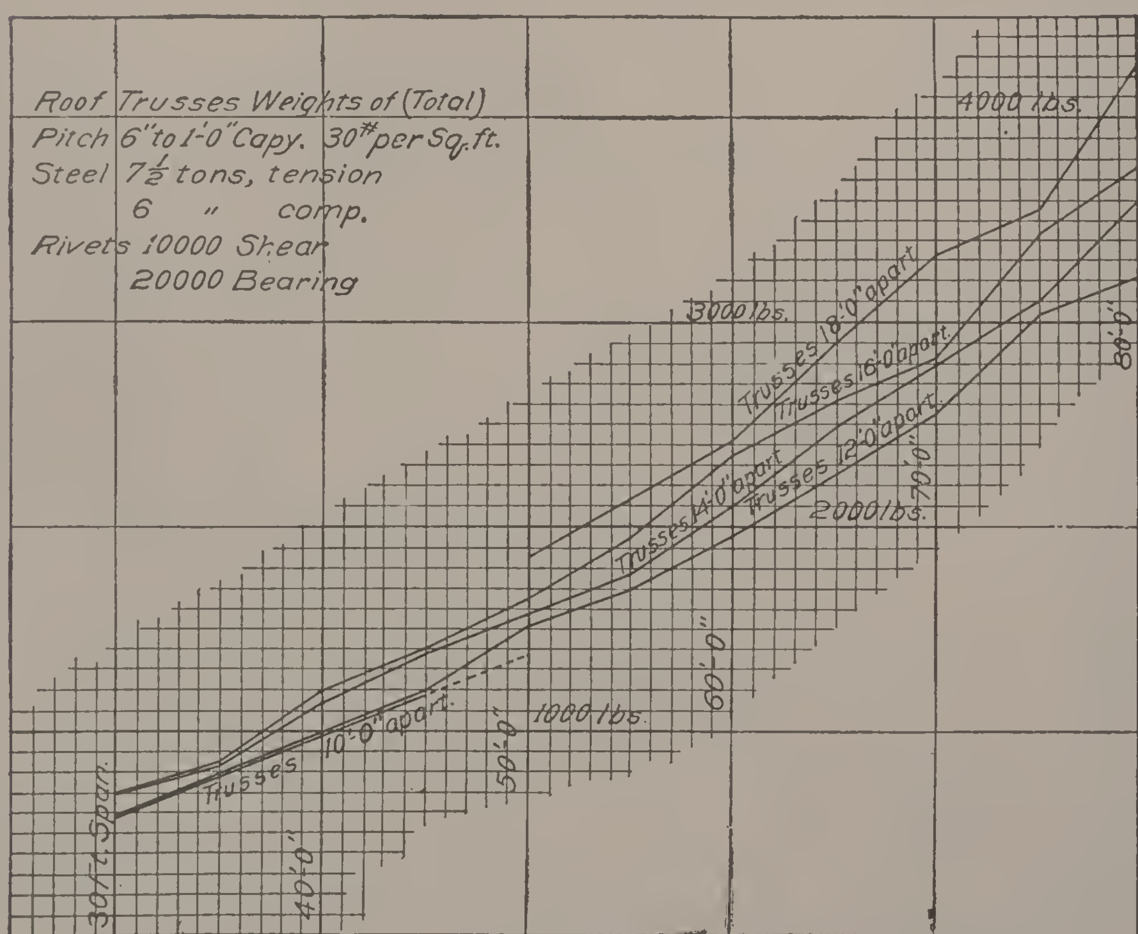


Fig. 237.

Columns-safe loads by formula $13750-577\frac{l}{r}$-Square Ends. 4 Ls laced. (12" b. to b.) long legs out. Medium Steel- Standard Angles. Dotted line shows limit for 45 diam. Full line for 70 diam.													
Size of Ang's	Area 4 Ls Sq. in.	Weight per Ft. 4 Ls.	r	Safe Load (lb) for Lengths in Feet of —									
				12	13	14	15	16	17	18	19	20	21
2" × 2½ × 3/16"	3.24	11.2	1.28	27000	25530	24050	22580	21110	19640	18170	16700	15220	13750
2" × 2½" × ¼"	4.24	14.8	1.29	35700	33800	31900	30000	28100	26200	24300	22400	20500	18600
2½" × 3" × ¼"	5.24	18.0	1.50	47800	45800	43750	41730	39710	37700	35670	33650	31620	29600
2½" × 3" × 5/16"	6.48	22.0	1.50	59300	56820	54340	51860	49390	46910	44430	41950	39480	37000
2½" × 3½" × ¼"	5.76	19.6	1.76	56300	54440	52590	50730	48880	47020	45170	43310	41450	39600
2½" × 3½" × 5/16"	7.12	24.4	1.76	69800	67490	65177	62870	60555	58250	55930	53620	51310	49000
2½" × 3½" × 3/8"	8.44	28.8	1.77	83700	80840	77980	75130	72300	69400	66600	63700	60900	58000
3" × 3½" × 5/16"	7.72	26.4	1.71	74800	72200	69660	67100	64500	61960	59400	56830	54270	51700
3" × 3½" × 3/8"	9.20	31.2	1.71	89200	86100	83000	79960	76900	73800	70700	67600	64500	61500
3" × 4" × 5/16"	8.36	28.4	1.97	85200	82800	80380	78000	75550	73140	70730	68320	65910	63500
3" × 4" × 3/8"	9.92	34.0	1.98	101000	98000	95100	92360	89600	86750	83930	81120	78310	75500
3" × 5" × 5/16"	9.60	32.8	2.52	105500	103300	101100	98900	96740	94550	92360	90180	88000	85800
3" × 5" × 3/8"	11.44	39.2	2.52	125800	123200	120600	118000	115400	112850	110260	107670	105080	102500

Fig. 238.

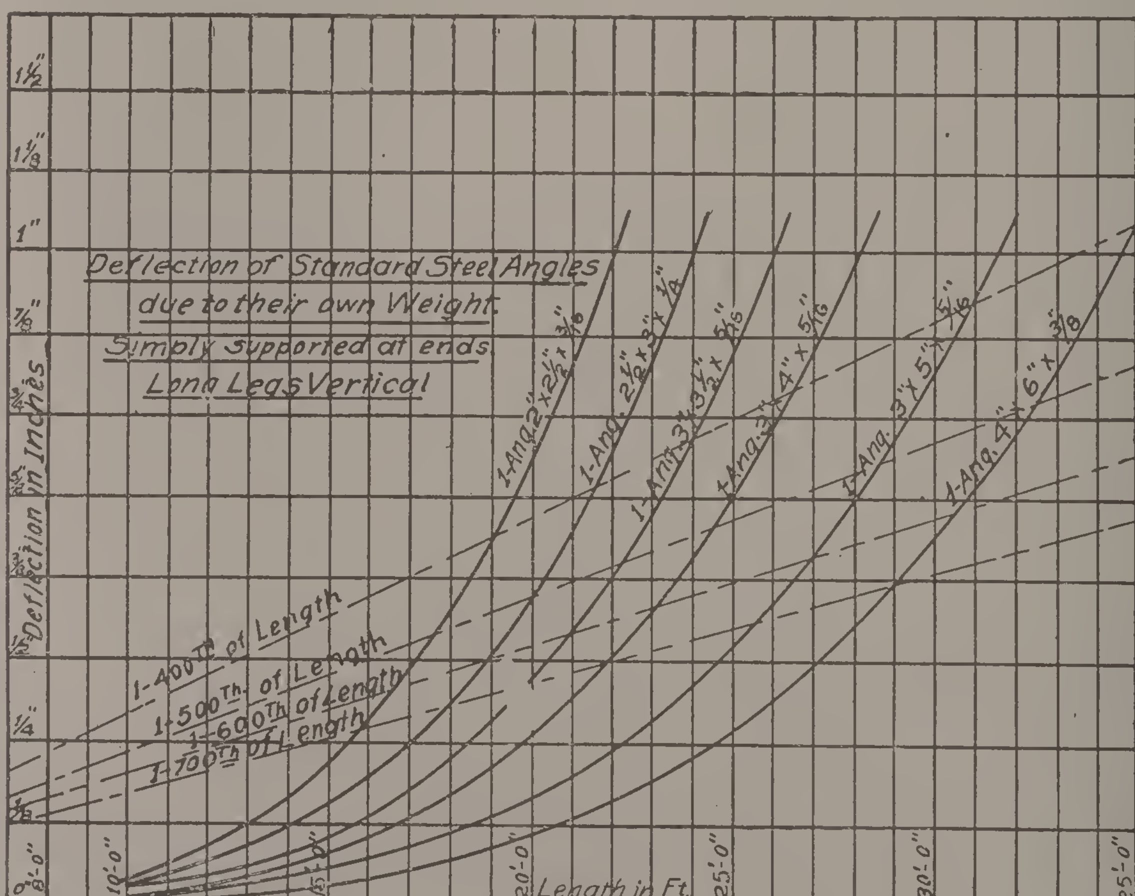


Fig. 239.

PROBLEMS.

1. Fig. 226 shows a framing plan on which is all the information necessary to detail the different members. Make a detail of column No. 4, assuming that the bottom of the column rests on a cast iron web base 12 ft. below the top of the 15-in. beam No. 9, and that the column is arranged to receive another column of the same size, the joint being 1 ft. 6 in. above the top of the 15-in. beam.

2. Make details covering the 10 to 13-inch beams in Fig. 208.

3. Make a schedule of tie rods and of field bolts required for all framing shown in Fig. 208.

4. Suppose that in Fig. 199 the cast iron columns had a 12-in., $31\frac{1}{2}$-lb. beam instead of the 15-in. beam, a 7-in. instead of the 10-in. beam, and a 9-in. instead of the 12-in. beam, and that all beams were flush on top and the other features the same as shown in Fig. 210. Make a detail of such a column.

5. Make a bill of material for the column shown in Fig. 224.

6. Given a portion of a framing plan as shown in Fig. 226. Make shop details of (*a*) beam No. 1 resting on the column. (*b*) beam No. 2, and (*c*) channels No. 3 and 4.

7. Given a beam box girder framed between two columns as shown in Fig. 227. Make a shop detail of this girder using a uniform pitch of rivets of 3 in. in the plates.

8. Given a lintel composed of two 10-in., 15-lb. channels framed between two columns, the channels being placed with the flanges turned in, 10 in. back to back, and 14 ft. 8 in. between the faces of columns. Make a complete shop detail and order for all parts.

9. Given a pit under an elevator to be framed with 3×3-in., 6.6-lb. Ts, 17-in. on centers, to receive terra cotta tile. These Ts frame between the webs of 15-in., 42-lb. I-beams at each end, which are 7 ft. 3 in. center to center. The distance from the top of the beams to the bottom of the flange of the Ts is 6 in. Make a shop detail of the Ts.

10. Given a portion of a framing plan as shown in Fig. 228. Make a shop detail of beams No. 1 and 2 which are framed between columns.

11. In the above plan make detail covering beams, No. 4 and 5.

12. Given a 15-in., 42-lb. beam framed between the webs of two columns. 20 ft. 8 in. center to center on a line perpendicular to

the axis of the webs, and the center of one column is 1 ft. 9 in. off from the other in the direction of the webs. The webs of the columns are $\frac{1}{2}$ in. thick. The beam has a 12-ft., $31\frac{1}{2}$-lb. beam framed to it, 2 ft. 1 in. from the center of one column, the tops being flush. There are also holes for two lines of $\frac{3}{4}$-in. tie rods. Make shop detail.

LA SALLE STATION, L. S. & M. S. AND C., R. I. & P. RAILROADS, CHICAGO

Frost & Granger, Architects; E. C. & R. M. Shankland, Engineers

Steel trusses over train shed; span of truss, 215 feet. Note the traveling crane, with three derricks on it, used in setting these trusses.

STEEL CONSTRUCTION

PART IV

RIVETED GIRDERS

Functions of Flanges and Web. Riveted girders are made up of two general parts (a)—the top and bottom members—which are termed, respectively, the *top* and *bottom flanges*; and one or more vertical plates (b), called the *web-plate*, connecting the top and bottom flanges.

Girders of one web-plate are called *single-web* girders; of two plates, *double-web* girders; of three plates, *triple-web* girders. Figs. 240, 241, and 242 illustrate these different types.

The function of the flanges is to take the compression and tensile stresses developed in the outer fibers by the beam action. The function of the web is to unite these two flanges and to take care of the

Fig. 240.

Fig. 241.

Fig. 242.

shear. These functions are distinct. In a rolled beam, the stresses are considered to be distributed over the whole cross-section just as in a rectangular wooden beam; and this stress varies uniformly from the neutral axis. A rolled beam, therefore, is proportioned by using the beam formula, and determining from it the required moment of inertia.

A riveted girder, however, is not a homegeneous section; the flanges are separate from the web, except as they are united to it at intervals by rivets. For this reason the stress in the extreme fibers on the compression and tension sides is considered as concentrated at the

center of gravity of the flange, and the flanges are considered as taking all the compression and tension stress.

The bending moments caused by the vertical loads acting on the girders are considered as resisted, therefore, by their tension and compression stresses, which form a couple whose arm is the distance between the centers of gravity of the two flanges, as illustrated by Fig. 243.

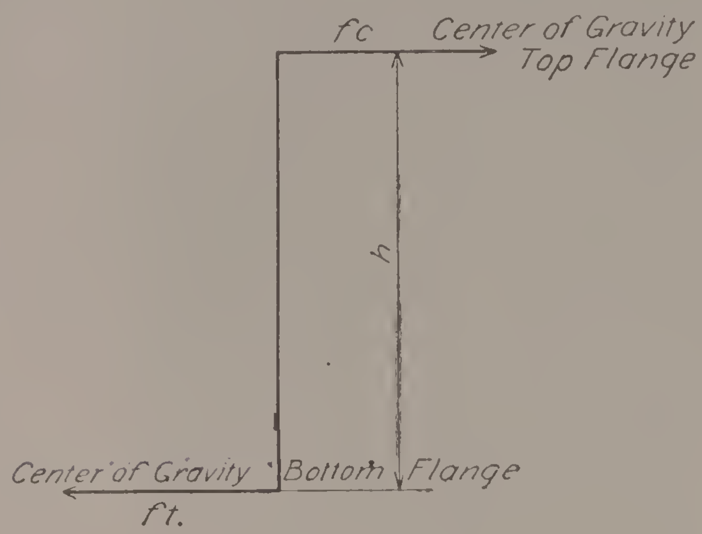


Fig. 243.

Proportioning Flanges. Referring to Fig. 244, if the bending moment on the girder

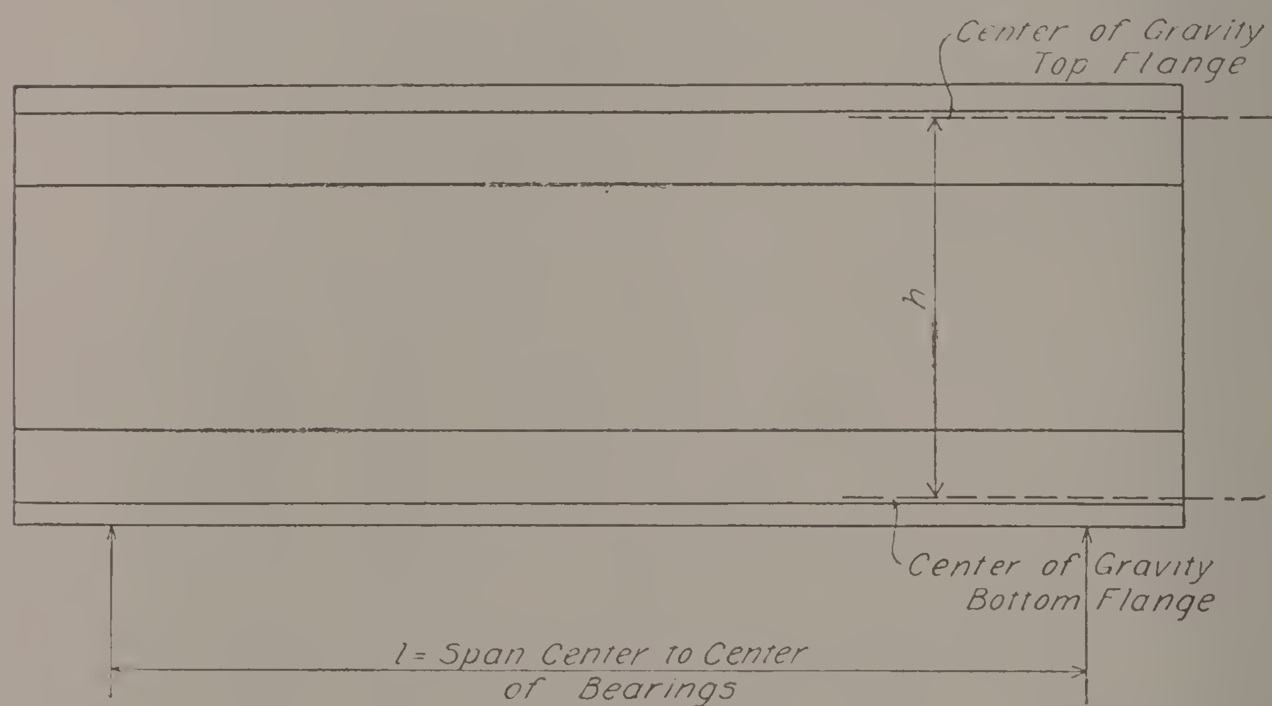


Fig. 244.

is M, and h is the distance between centers of gravity of flanges then

$\frac{M}{h} = F =$ the tension and compression forces necessary to balance the bending moment.

If $f_c =$ Allowable Stress per square inch in *compression*, and if

$f_t =$ Allowable Stress per square inch in *tension*; then

$\frac{F}{f_c} =$ Area required in *compression* flanges, and

$\frac{F}{f_t} =$ Area required in *tension* flange.

The values of f_c and f_t vary with the class of construction in which the girders are used. These are generally specified in each case. The usual values for different classes of construction are as follows:

ALLOWABLE VALUES

For Buildings:

f_t (tension) = 15,000 pounds per square inch, net area.

f_c (compression) = 12,000 pounds per square inch, gross area, reduced for ratio of unsupported length to width of flange.

f_s (shearing stress) = 12,000 pounds per square inch, net area.

For Highway Bridges:

f_t = 13,000 pounds per square inch, net area.

f_c = 11,000 pounds per square inch, gross area, reduced for ratio of length to width of flange.

f_s = 10,000 pounds per square inch, net area.

For Railway Bridges:

f_t = 10,000 pounds per square inch, net area.

f_c = 8,000 pounds per square inch, gross area, reduced for ratio of length to width of flange.

f_s = 8,000 pounds per square inch net area.

The practice regarding the reduction of allowable compression stress varies somewhat; but the following formula is a conservative one for general use:

$$f = \frac{f_c}{1 + \frac{l^2}{5{,}000\ W^2}},$$

where f = Fiber stress to be used in compression;
f_c = Specified fiber stress unreduced;
l = Length of unsupported flange (in inches);
W = Width of flange (in inches).

In ordinary construction, the fact that the two flanges are generally made of the same section makes it unnecessary in many instances to consider this reduced compression-fiber stress. If the unsupported length of top flange is long, however, so as to make the section determined for bottom flange insufficient, then both flanges should be made the same as that required by the compression value.

When the girder is short, and the web-plate is not spliced, allowance is sometimes made for the portion of the compression and tension

which the web may carry. In doing this, the net area of the web—deducting rivet-holes—is considered concentrated at the centers of gravity of the flanges, and as reducing the required area of the flanges by an amount equal to $\frac{1}{8}\,t\,h_1$, in which t = thickness of web, and h_1 = depth. When this assumption is made, therefore, the required area of each flange is $\frac{F}{f} - \frac{1}{8}\,t\,h_1$, in which f is the compression value for the top flange and the tension value for the bottom flange.

There is a considerable saving in the templet and shop work if both flanges are made alike; the extra weight in one flange which may be added, will often be more than offset by the saving in shop work.

It is a very general practice, therefore, to make both flanges alike in section, determining this by whichever flange requires to be the larger.

Economical Depth of Web. It will be seen that the areas required for the flanges are dependent on the depth of the web. Where there are no conditions limiting this depth to certain values, it is desirable, therefore, to fix it so as to give the most economical section. For a uniformly distributed load, this depth is generally from $\frac{1}{9}$ to $\frac{1}{10}$ of the span. Sometimes several approximations of this depth can be made, and the corresponding areas determined; and then, by computing the weights of flanges and web-plates so determined, the most economical section can be chosen.

In a great many cases, especially in building construction, the economical depth cannot be used, because of conditions fixing this depth with relation to other portions of the construction. In other cases, certain sections of plates and angles must be used in order to obtain quick delivery; and accordingly, the depth must be fixed to harmonize with these sections.

Proportioning the Web. As before stated, the function of the web is to resist the shear.

The student should here note that, as explained under "Statics," the loading which will produce maximum shear is not necessarily the same as that which causes the maximum bending moment.

In highway and railway girders, this loading is always different. In building construction it is very often different, because certain beams may frame into the girder over the support and these beams must be considered in determining the shear although they are not con-

sidered in determining the bending moment. Again, a girder may carry a wall, and a portion of this wall may come directly over the end supports of the girder. This portion will materially increase the shear while perhaps not affecting the bending moment.

The general statement of loads to be considered in determining the shear where all loads are fixed in position, is to include all loads which directly or indirectly can come upon the girder, and to determine the maximum end reaction for these loads. (The determination of web shear for moving loads, will be treated under "Bridge Engineering).'' Sometimes the shear at one end is greater than at the other, in which case the section is fixed by the requirements at the end having greatest shear.

Having determined therefore, the maximum shear, the required area of web is

$$\frac{S}{f_s} = \tfrac{3}{4}\, t\, h$$

in which S = Maximum shear;

f_s = Allowable shearing stress per square inch of net area of web;

t = Thickness of web; and

h = Depth of web.

The net area is assumed as $\frac{3}{4}$ the gross area.

Crippling of Web, and Use of Stiffeners. The value of f_s to be used depends on the clear distance between the adjacent edges of the top and bottom flange angles, and upon whether or not stiffener angles are to be used.

The distribution of the shear over the web causes compression forces acting at angles of 45 degrees with the axis of the girder, in the manner indicated by Fig. 245. The web, therefore, under these compression stresses, is subject to failure laterally, just as a long column. The allowable shearing stress must therefore be reduced by a formula similar to the column formula, which may be taken as

$$f_s = \frac{12{,}000}{1 + \dfrac{d\,c^2}{3{,}000\,t^2}},$$

in which $d\,c$ = distance between flanges; and t = thickness of web.

Either the web must be made thick enough not to exceed this allowable stress on a length 1,414 $d\,c$, which is the length on a 45-degree line between the adjacent edges of flange angles, or this unsupported

length must be reduced by using stiffeners so spaced as to cut this 45-degree length down to limits which will conform to the allowable shearing stress given by the formula and to the thickness of web which it is desired to use.

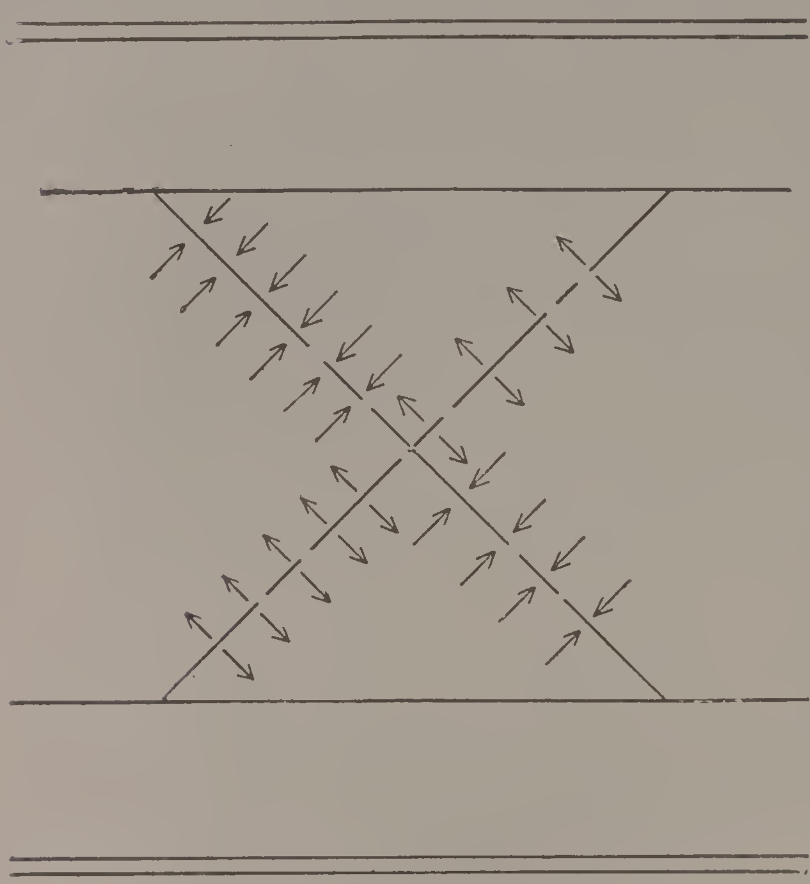


Fig. 245.

Webs less than $\frac{5}{16}$ inch thick are rarely used. For greater thicknesses, it is a matter of economy generally to use stiffeners. For very heavy loads, however, or for long spans, $\frac{3}{8}$-inch or $\frac{1}{2}$-inch webs would be used, with or without stiffeners, as might be required.

It will be seen from the above consideration, that, where the shear varies from the end towards the center, the required spacing of stiffeners will increase towards the center, since the area of the web is constant.

When the shear has reduced to the point where the area of web is sufficient to resist buckling on a length of 1.414 d c, then the stiffeners may be omitted. A convenient diagram for determining spacing of stiffeners is shown in Fig. 246; the use of this diagram will be illustrated by a problem.

Suppose the shear at the end of a girder is 100,000 pounds; and the clear distance between flange angles is 22 inches, and the web which it is desired to use is 30 inches by $\frac{3}{8}$ inch. The gross area of web is then 11.25 square inches, and the shear per square inch of gross area is 8,900 pounds. Following up the vertical side of the diagram until the line corresponding to 8,900 is found, then following this line until it meets the line of a $\frac{3}{8}$-inch web, and then looking under this intersection to the lower horizontal line, it is found that stiffeners must be spaced about 12 inches apart in order to conform to the above conditions.

If it was desired to find what thickness of web was necessary in

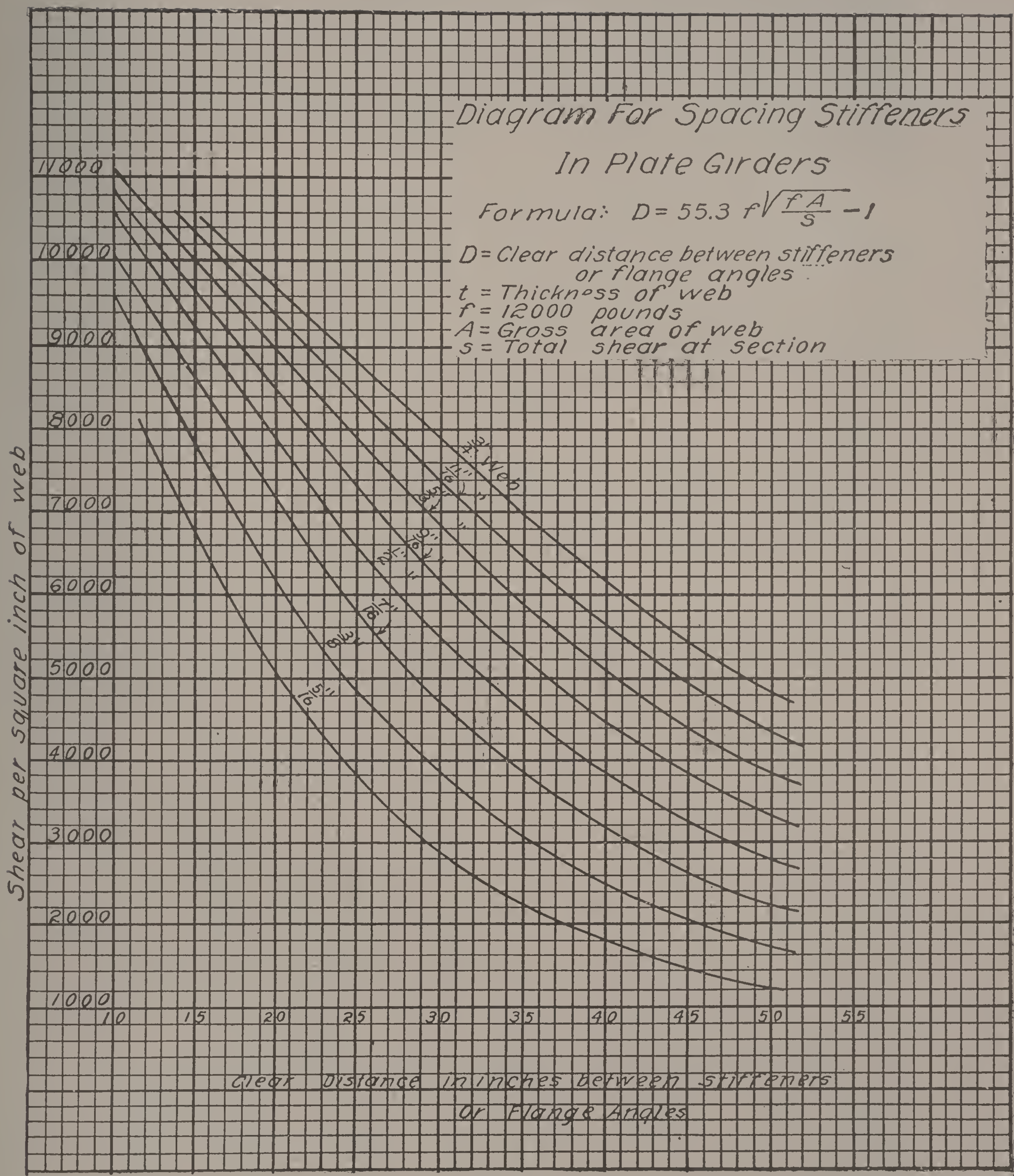


Diagram For Spacing Stiffeners
In Plate Girders
Formula: $D = 55.3\ f\sqrt{\frac{fA}{s} - 1}$
D = Clear distance between stiffeners or flange angles
t = Thickness of web
f = 12000 pounds
A = Gross area of web
s = Total shear at section
Shear per square inch of web
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
10
15
20
25
30
35
40
45
50
55
Clear Distance in inches between stiffeners or Flange Angles
$\frac{3}{4}$" web
$\frac{5}{16}$"
$\frac{3}{8}$"
$\frac{7}{16}$"
$\frac{1}{2}$"
$\frac{9}{16}$"
$\frac{5}{8}$"
$\frac{11}{16}$"

Fig. 246.

order not to require stiffeners, the flange angles being 22 inches apart in the clear, this would be determined as follows:

Follow up the vertical line corresponding to 22 inches as given at the bottom of the diagram, until this line meets the line corresponding to such a thickness of web that the gross area is sufficient to bring the shearing stress within the limit by the horizontal line at this intersection of web-line and vertical through 22.

In this case the nearest intersection is found to be the $\frac{1}{2}$-inch web. The area of a 30-inch by $\frac{1}{2}$-inch web is 15 square inches, and this gives a shearing stress per square inch of 6,675 pounds. The allowable stress as given by the diagram is 7,400 pounds; but the $\frac{7}{16}$-inch web found to give a shearing stress of 7,640 pounds, whereas the allowable shear for a $\frac{7}{16}$-inch web with angles 22 inches apart is only 6,600 pounds.

It would be found more economical to use a $\frac{3}{8}$-inch web with stiffeners, than a $\frac{1}{2}$-inch web without stiffeners.

Another use of stiffeners is to stiffen the web at concentrated loads. The most important case under this head is the reaction at the bearings of the girder. Stiffeners are always used here, and they are generally placed so that the outstanding legs will come nearly over the edge of the bearing plate, as illustrated by Fig. 247. Sometimes

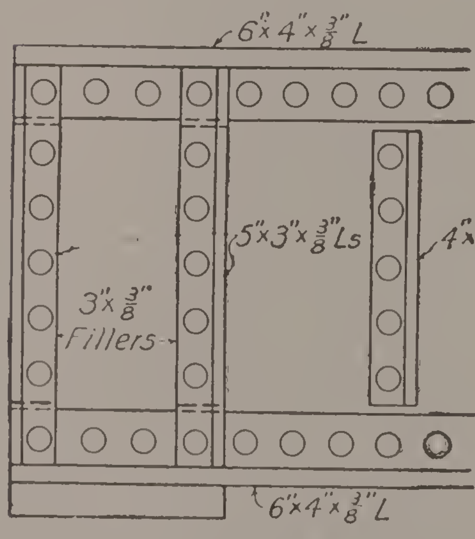


Fig. 247.

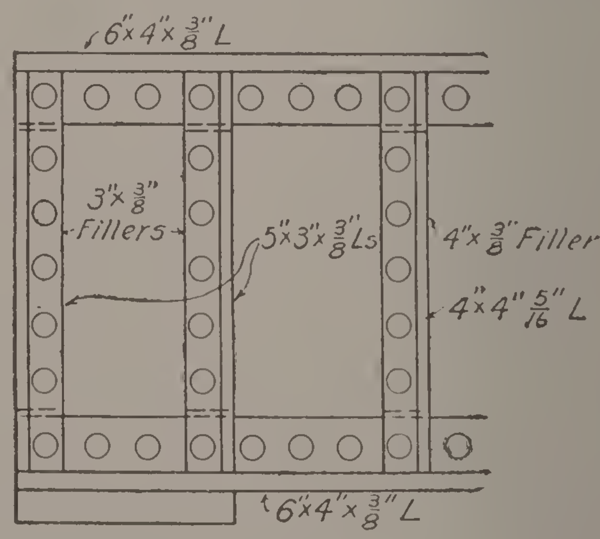


Fig. 248.

the special nature of the bearing—as, for instance, the disposition of column members—makes it desirable to place these stiffeners close together, or in three lines instead of two. The fundamental idea is to place the stiffeners so as to distribute the reaction in the most direct way to the bearing. If this bearing is masonry, the stiffeners will be placed so as to give uniform bearing; if a column, they will be placed so as to correspond as closely as possible with the line members of

the column. Wherever heavy concentrated loads from beams, other girders, masonry piers, etc., occur, stiffeners should be used to stiffen the web against this concentrated application of load. Stiffeners over bearings should be fitted to both the top and bottom flange angles. Stiffeners at loads on the top flange need be fitted only to the top flange angles.

Stiffeners used simply to prevent buckling from the shear, need not be fitted to either flange. Sometimes stiffeners used for this latter purpose are not carried over the flange angles, but stop clear so as to avoid the necessity of fillers, as indicated by Fig. 247. It is better practice, and more generally followed, to carry these angles over the flange angles, as shown by Fig. 248.

PROBLEMS

1. Determine by the method previously described the bottom flange section of a girder 28 inches deep between centers of gravity of flanges, and having a bending moment of 3,500,000 inch-pounds. The flange is to be proportioned to carry the whole bending moment. Use fiber stresses given for building.

2. In the above problem, if the top flange is unsupported laterally for 20 feet, determine the section of top flange required, using the formula given for reducing allowable compression stress.

3. Given a girder 35 feet long between centers of bearings, and carrying a uniformly distributed load of 2,000 pounds per linear foot. Assume a web 36 inches deep and 34 inches between centers of gravity of flanges. Determine bottom flange section without making any allowance for the portion of bending moment carried by the web.

4. In the above girder, redesign bottom section on the basis that the web is not spliced and that it bears a portion of the bending moment.

5. Determine the thickness of web required in above girder.

6. If the girder was 40 feet long, 42 inches deep, and loaded with 4,000 pounds per linear foot, determine the thickness of web if no stiffeners are to be used. Assume flange angles are 6 inches by 6 inches by $\frac{1}{2}$ inch.

7. Determine thickness of web in above girder which could be used with stiffeners, and determine spacing of stiffeners required.

Solution. In this case the shear at end is 80,000 pounds. From the diagram for spacing of stiffeners, it will be seen that any thickness

of web from $\frac{5}{16}$ inch up could be used. Where stiffeners are used to prevent buckling of web, it is more economical to use a $\frac{5}{16}$-inch web than a $\frac{3}{8}$-inch. If the girder was 60 inches deep, probably it would not be well to use less than $\frac{3}{8}$-inch web, even with stiffeners. In this case assume a $\frac{5}{16}$ by 42-inch web. Area is therefore 13.12 square inches, and fiber stress is 6,150 pounds.

From the diagram it is seen that a $\frac{5}{16}$-inch web with this stress per square inch requires stiffeners about $16\frac{1}{2}$ inches back to back. This then determines the space of first stiffener from those over the bearing plate. Assume two spaces the same as this, and then determine shear at point say 3 feet 6 inches from the end bearing. This is found to be $80,000 - (4,000 \times 3.5) = 66,000$ pounds. The stress here is about 5,075 pounds per square inch of web. From the diagram, this is seen to require stiffeners 20 inches apart. Assume two more spaces at 20 inches, and calculate shear, which is found to be 52,600 pounds. This gives a fiber stress of 4,050 pounds per square inch of web, and requires stiffeners 24 inches apart. Take three spaces at this distance, and calculate shear, which is found at this point to be 28,600. This gives a stress of 2,200 pounds per square inch of web. From the diagram the spacing of stiffeners for this fiber stress, in a $\frac{5}{16}$-inch web, is found to be 36 inches. This distance, however, is greater than the clear distance between flange angles, which is 30 inches, and indicates, therefore, that at this point the web is strong enough without being stiffened by angles.

If it is desired to see whether or not two spaces at 24 inches, instead of three as above taken, would have been sufficient, the shear at this point can be calculated. This is found to be 36,600 pounds, or 2,800 pounds per square inch of web. This is seen to require stiffeners 31 inches apart. This is greater than the distance between flange angles, and indicates that the last stiffener could be omitted. However, it is better to carry the stiffeners a little beyond the actual point where the diagram would indicate that they could be dropped; so that it would be better to use the last stiffener, as originally determined. The spacing of stiffeners at each end of girder is of course made the same where the load is uniformly distributed.

Size of Stiffeners. Stiffeners for concentrated loads and for reactions should have sufficient area to take the whole load or reaction at this point. Stiffeners used to prevent buckling are not generally

calculated, but are made either 3 x 3 x $\frac{3}{8}$ inch or 4 x 3 x $\frac{3}{8}$ inch. When stiffeners are fitted to the flanges, the outstanding leg should be made large enough to come nearly out to the edge of the flange angle. If the flange angle is 6 by 6, the stiffener would be perhaps 5 by $3\frac{1}{2}$.

Cutting Off Flange Plate. In heavy girders the flanges are made of angles with cover-plate. Sometimes only one plate is required; at other times four or more will be needed. As the maximum moment is the moment determining the flange section, and this usually varies from point to point, it will be seen that for economy the number of plates should be proportioned to the varying moment. Where the girder is loaded uniformly, the bending moment is a maximum at the center of the span, and varies toward the ends as the offsets to a parabola. A convenient way, therefore, to determine for such a case where to stop the different plates, is to lay off to scale the span, and on this axis construct a parabola, making the ordinate at the center represent the required area, from the formula $\frac{M}{fh} = A$. A convenient method of constructing the parabola will be to lay off the offsets, which are determined at different points by the formula

$$y = h\left[1-\left(\frac{x}{\frac{l}{2}}\right)^2\right], \text{ as illustrated by Fig. 249.}$$

From this diagram, the point at which an area equal to one of the plates can be dropped off, will be found by drawing a horizontal line at a distance down equal to the area of the plate at the same scale as the center ordinate. Where this line cuts the line of the parabola, will be the exact length of plate required. Sufficient length should be added at each end to enable rivets enough to be used to develop in single shear the stress in the plate. Usually this will be about 1 foot 6 inches at each end.

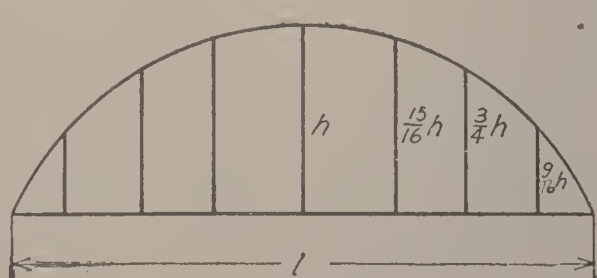


Fig. 249.

Another method of determining where to drop off plates when the load is uniformly distributed, is to use the formula

$$x = \tfrac{1}{2}\sqrt{\frac{A_1 l^2}{A}},$$

in which x = Distance from center to point where area of plate is not required;

A_1 = Area of plate to be cut off;

A = Total required flange area at center,

$= \frac{M}{f h}$; and

L = Span.

When the loads are concentrated, and the moment does not vary uniformly from point to point, the only way is to calculate the moment at different points, and proportion the flange and at these points in the same manner as at the center.

PROBLEMS

1. Given a girder 50 feet long, having a flange section of two angles 6 x 4 x $\frac{1}{2}$, and 2 cover-plates 10 x $\frac{3}{8}$ inch. Construct a parabola on this length as an axis, and determine the distances between the points where from diagram each cover-plate could be left off.

2. In above girder, determine actual length of cover-plates required by using the formula for cutting off plates.

3. Given a girder 40 feet long between centers of bearing, loaded with 120,000 pounds concentrated at four points equally distant. Determine the bottom flange section, and length of cover-plates.

Solution. Max M. = 30,000 × 8 × 3 × 12 = 8,640,000 inch-pounds. Assume web 36 inches deep, and effective depth as 34 inches; then flange stress = 254,000 pounds. This, at 15,000 pounds' fiber stress, requires $\frac{254{,}000}{15{,}000}$ = 16.95 square inches.

In this, as in all calculations of girders, a great many sections could be chosen. In all problems the student must use his own judgment as to just what shapes to use in order to make up the section. Take

2 angles 6 x 6 x $\frac{3}{8}$	= 7.28	(two holes out)
2 plates 14 x $\frac{3}{8}$	= 9.75	(two holes out)
	17.03	

Note that in deducting area of rivet-holes from bottom flange, the hole is considered 1 inch in diameter, even though $\frac{3}{4}$-inch rivets are used. If smaller rivets were used, this might reduce the assumed diameter of hole to $\frac{3}{4}$ inch.

From the manner in which this girder is loaded, it will be seen that the points at which the plate can be left off will be near the concentrated loads. Omitting both plates will leave a net area of 7.28 square inches; this corresponds to a flange stress of $7.28 \times 15{,}000 = 109{,}200$ pounds; and to a bending moment, assuming the same effective depth as at the center, of $109{,}200 \times 34 = 3{,}712{,}800$ inch-pounds. The reaction is 60,000 pounds; and it is therefore seen that the point corresponding to this moment is between the reaction and the first load. Its position is found as $\frac{3{,}712{,}800}{60{,}000} = 61.88$ inches = 5 feet $1\frac{7}{8}$ inches.

If this first plate is carried 1 foot 6 inches beyond this point, then its total length becomes 32 feet $7\frac{1}{2}$ inches.

At the point where the second plate is dropped, the net area is 12.10 square inches. This corresponds to a flange stress of $12.10 \times 15{,}000 = 181{,}500$ pounds; and to a bending moment of $181{,}500 \times 34 = 6{,}160{,}000$ inch-pounds.

The bending moment at the load nearest the reaction is $60{,}000 \times 8 \times 12 = 5{,}760{,}000$ inch-pounds.

The moment between this load and the next load increases by an amount equal to $60{,}000 - 30{,}000$, multiplied by the distance from the load. That is, at a point x distance from the last load, the moment will have increased $(60{,}000 - 30{,}000) \times x \times 12$ inch-pounds.

The bending moment which the angles and one cover-plate can carry was found to be 6,160,000 inch-pounds. The moment at first load is $\frac{5{,}760{,}000}{400{,}000}$ = allowable increase to point where second cover is required.

The distance from this first load to the point where it will be necessary to add the second cover-plate, is found, therefore, to be

$$\frac{400{,}000}{30{,}000 \times 12} = 1.12 \text{ feet.}$$

As this is so near the point at which the load is applied, it would be better to add a little more than 1 foot 6 inches to this distance, in order to carry the plate a little beyond where the concentrated load occurs. This would make it necessary to increase slightly the length of the first cover from what was previously determined. These plates might be fixed, therefore, as 26 feet long and 34 feet long, respectively.

Spacing of Flange Rivets. The purpose of the rivets through the flange is to provide for the horizontal shear. There is a definite relation between the horizontal shear and the vertical shear at a given point, which is expressed by the formula $s = \frac{S\,Q}{I}$, in which

s = Horizontal shear per linear inch;
S = Total vertical shear at section;
Q = Statical moment of the flange about the neutral axis of the girder; and
I = the moment of inertia of the whole section of the girder about its neutral axis.

Having determined the horizontal shear per linear inch, the spacing becomes the value of one rivet divided by this horizontal unit shear, or

$$d = \frac{V}{s}.$$

For the vertical rivets through flange angles and cover-plates, the same formula applies, except that Q is taken as the statical moment of the cover-plates only about the neutral axis.

The above exact method is not the one generally followed in spacing rivets, because it is not generally necessary to space the rivets so nearly to the exact theoretical distance. It is quite a common custom to space these horizontal flange rivets by assuming that the horizontal shear is equal to the vertical shear at the section divided by the distance between the centers of gravity of the flanges. This gives spaces somewhat less than would be required by the formula above.

The vertical rivets through cover-plates are made to alternate with the horizontal rivets; and in general, if there are sufficient horizontal rivets, this method will give sufficient vertical rivets. In doubtful cases, the exact method should be used.

It is customary to vary the spacing of the rivets about every two or three feet, or, in long girders, at intervals somewhat greater. This involves, of course, the determination of the shear at each point where a change in pitch is made.

The minimum distance in a straight line between rivets is three times the diameter of the rivet; if $\frac{3}{4}$-inch rivets are used, the minimum distance, therefore, is $2\frac{1}{4}$ inches. This is shown by Fig. 250. This fact many times determines the size of flange angles to be used. In

some cases the horizontal shear determining the pitch of rivets is so great that the distance between rivets becomes less than three times the diameter of the rivet. The flange stress might make it possible to use perhaps an angle with a 4-inch leg; in order to get in rivets

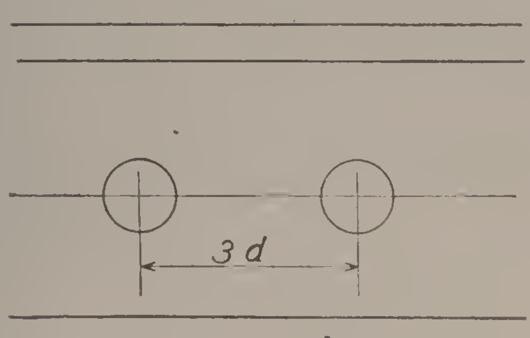


Fig. 250.

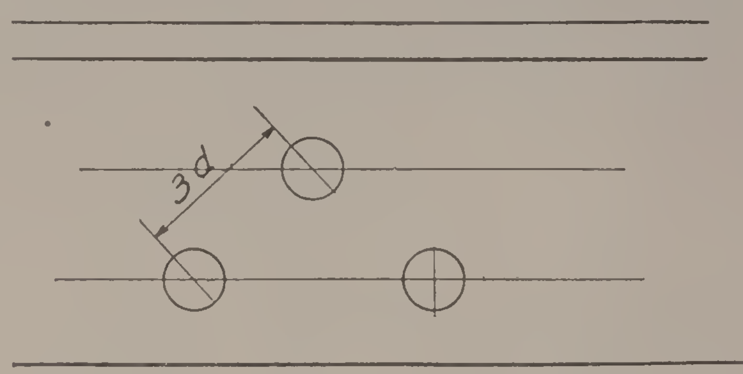


Fig. 251.

enough to take the shear, however, it becomes necessary to use an angle having a 6-inch leg so as to use two lines of rivets. In such a case the horizontal distance between center lines of rivets may be $1\frac{1}{4}$ inches, and still the direct distance between the rivets will not be under $2\frac{1}{4}$ inches. Fig. 251 illustrates this.

PROBLEMS

1. Determine the pitch at end of girder having a reaction of 60,000 pounds, with web-plate 30 inches deep and $\frac{3}{8}$ inch thick.

Assume effective depth between center of gravity of flanges, 28 inches; then approximate horizontal shear per linear inch $= \frac{60,000}{28} =$ 2,142.

The bearing value of a $\frac{3}{4}$-inch rivet on $\frac{3}{8}$-inch plate is 5,060; therefore pitch $= \frac{5,060}{2,142} = 2.35$ or $2\frac{5}{16}$ inches.

2. Given the same web as above, with an end reaction of 95,000 pounds, determine pitch at end.

Here $\frac{95,000}{28} = 3,400 =$ Horizontal shear per linear inch; and $\frac{5,060}{3,400} = 1.49$ or $1\frac{1}{2}$ inches.

This makes it necessary to use an angle deep enough to give two lines of rivets either a 5-inch or a 6-inch leg. If the pitch between rivet lines is $2\frac{1}{4}$ inches, and horizontally between rivets $1\frac{1}{2}$ inches, then the actual distance between rivets is about $2\frac{1}{16}$ inches, which is more than three times the diameter of the rivet. Where the top flange

of a girder is loaded directly, as by a heavy wall, it becomes necessary to calculate the rivets for direct shear as well as horizontal shear. The combined stress on the rivet must not exceed its value, and therefore a spacing somewhat less than that determined for horizontal shear above must be used. This can best be illustrated by a problem.

3. Given a girder having a web-plate 36 inches by $\frac{3}{8}$ inch, with an end reaction of 75,000 pounds, and loaded directly on top flange with 3,000 pounds per foot of girder, $\frac{75{,}000}{34} = 2{,}206$ = horizontal shear per inch. Assume a pitch of $2\frac{1}{4}$ inches; then

$2{,}206 \times 2.25 = 4{,}963$ = Horizontal stress on rivet;

$\frac{3{,}000}{12} = 250$ = Direct vertical shearing force per inch, and

$250 \times 2.25 = 560$ = Direct vertical load on rivet.

These forces act on the rivet as indicated by Fig. 252. The resultant, therefore, is the square root of the sum of the squares of these two forces, and equals 4,994. As the value of the rivet is 5,060, this is about the nearest even pitch which could be used for these combined stresses.

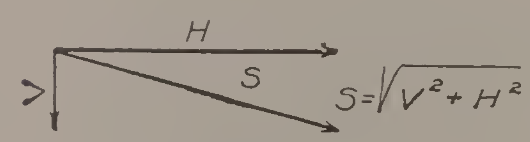


Fig. 252.

The maximum straight distance between rivets which can be used is 6 inches, or sixteen times the thickness of the thinnest metal riveted. For a flange having $\frac{5}{16}$-inch angles, therefore, 5 inches would be the maximum pitch; or, if a $\frac{1}{4}$-inch cover-plate were used, 4 inches would be the maximum in rivets through these cover-plates.

Vertical spacing of rivets in stiffeners does not generally require calculation. For end stiffeners there should be at least sufficient to take up all the end shear. In other stiffeners the pitch is generally made $2\frac{1}{2}$ or 3 inches.

Flange Splices. In long girders it becomes necessary sometimes to splice the flange angle and cover-plates. Sometimes, for purposes of shipment or erection, the girder has to be made in two or more parts and spliced.

In splicing the angles, the full capacity of the angles should be provided in the splice, regardless of whether the splice is at a point of maximum flange stress or not; it preferably should not be so located. Angles are used on either side of the flange angles, with the corner

rounded to fit accurately the fillet of the flange angle, and having the same gross or net area as these angles.

Fig. 253 shows the splice of the top flange of a plate girder. Note that the angles are spliced by cover-angles and also by cover-plates. In flange splices, provision should be made as far as possible to splice each leg of the angles directly and with sufficient rivets to provide

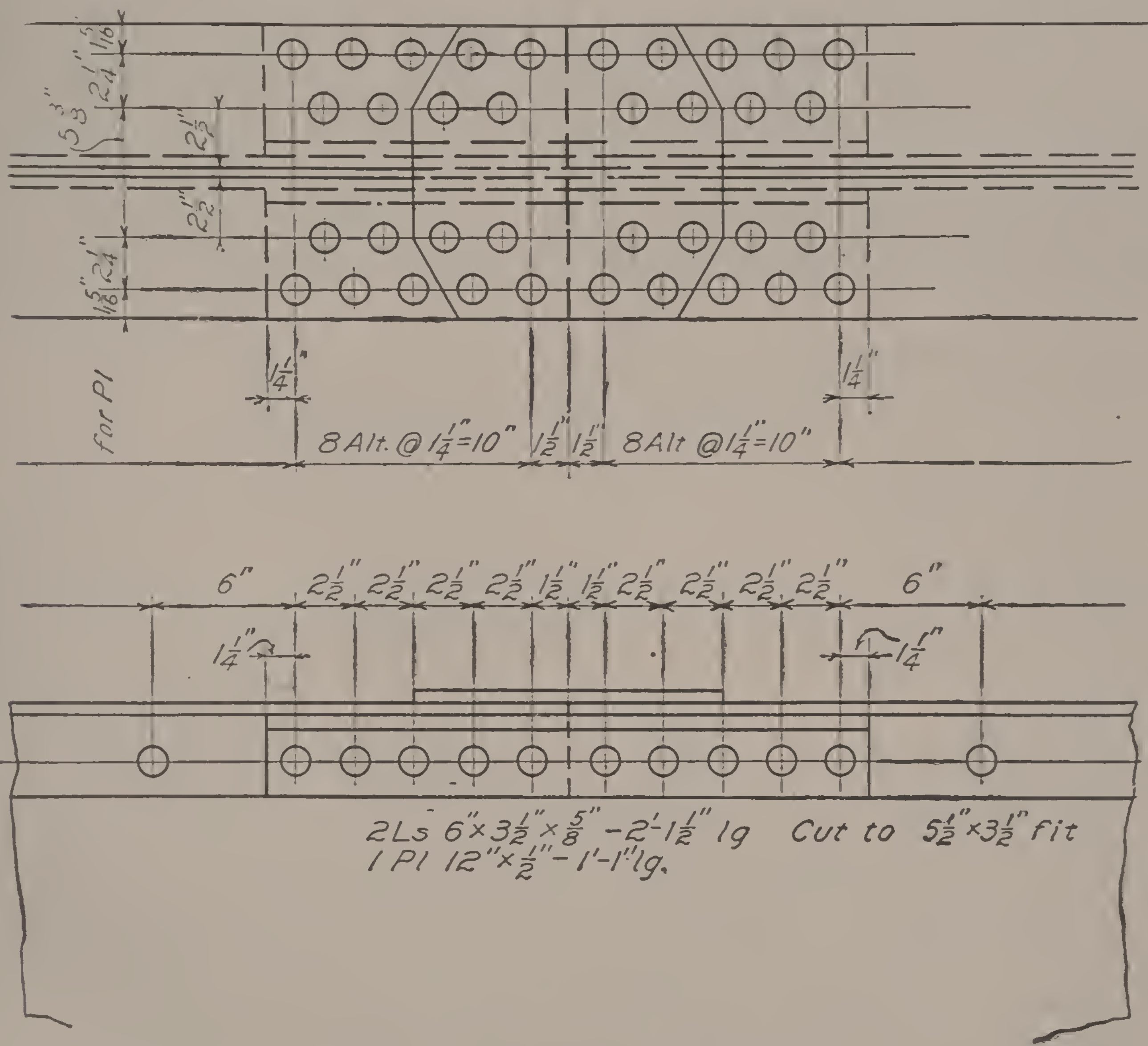


Fig. 253.

for the proportional part of the stress carried by this leg. If cover-plates form part of the section of the flange, these should, if possible, be spliced at a point where one of the cover-plates is not required for sectional area, and then this cover-plate should be carried far enough beyond the splice to provide rivets sufficient for the stress in the plate. If the plates are of different areas, an additional, short cover-plate over the splice would be required, to make up the required area.

When the shop work has to be very exact and reliable, a planed joint is sometimes used to take a portion of the stress by direct compression between the abutting ends. In such cases the cover angles

should be used, but may be of slightly less area. It is preferable, when possible, however, to have the flange fully spliced without relying on the planed joint. The number of rivets should be sufficient to provide for the full capacity of the flange angles without exceeding the value of a rivet. If one portion of the splice is hand-riveted, the values must be determined accordingly. Rivets are in double shear or bearing on the angles.

Web Splices. If the girder has been designed without considering that the web carries part of the flange stress, then the web splice need have only sufficient rivets to provide for the shear. If the web were considered as helping to carry the stress due to bending moment, then the splice would have to have sufficient rivets to resist this portion of

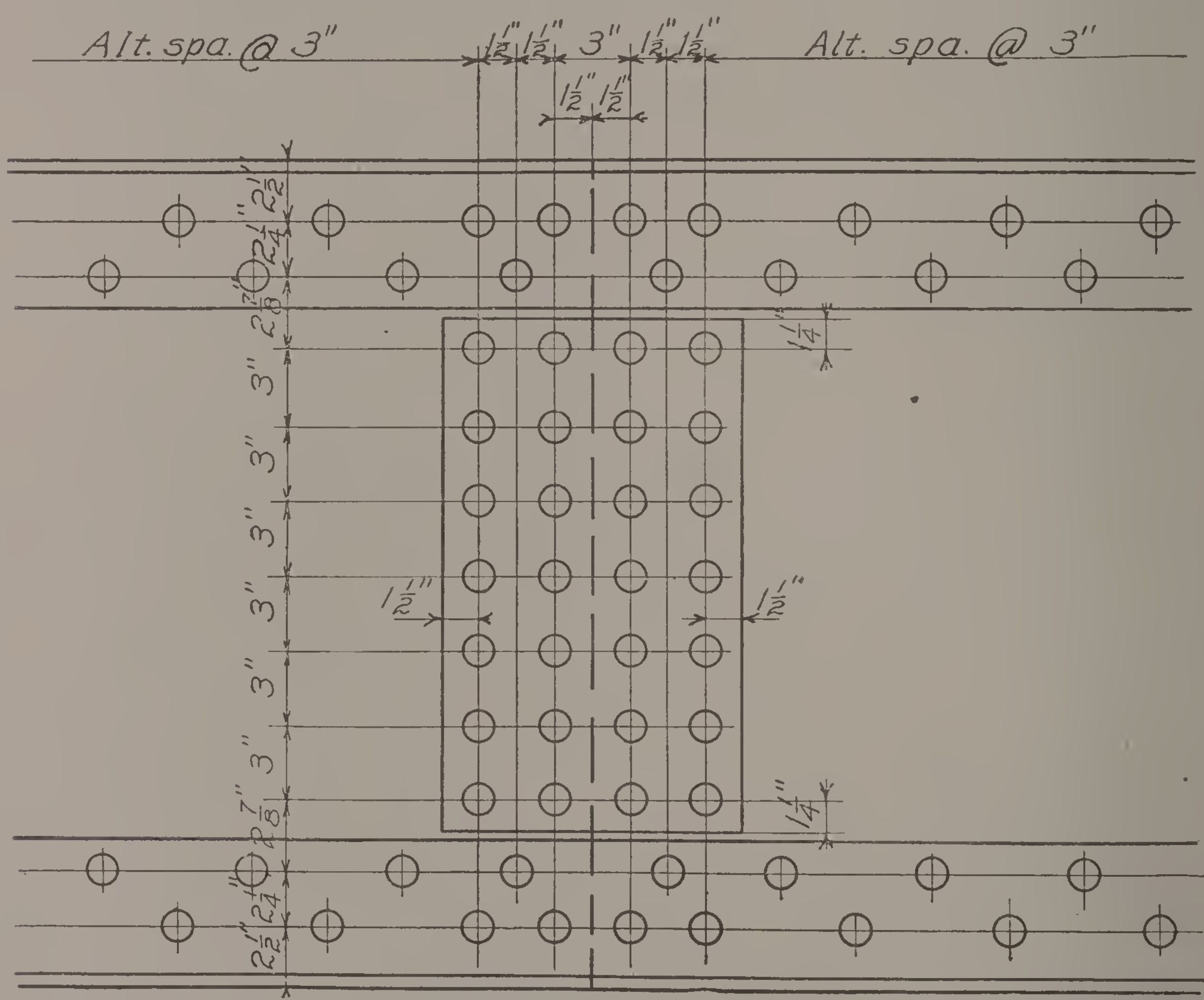


Fig. 254.

the bending moment carried by the web. In such a case, if two lines of rivets each side of the splice are used, and these rivets are spaced $2\frac{1}{2}$ or 3 inches center to center, they will be sufficient to provide for the shear and the bending moment also. In general it is better to use such a splice as illustrated in Fig. 254, whether the intention is to provide for bending moment or not.

The splice plates should have a net area equal to or a little greater than the net area of the web. If possible, the splice should be located at a point where the flanges are not fully stressed, so that they can help to splice the web.

PROBLEMS

1. As an illustration of the use of the exact formula for pitch of rivets, the following problem will be worked out:

Take the girder given in the problem illustrating the cutting-off of flange plate. This girder 40 feet long has a web-plate 36 inches by $\frac{3}{8}$ inch; and section of flange at end consists of two angles 6 x 6 x $\frac{3}{8}$ inch. At point 10 feet from end section, are two angles 6 x 6 x $\frac{3}{8}$ inch, and 2 plates 14 x $\frac{3}{8}$ inch.

Determine first the pitch of rivets at end where the shear is 60,000 pounds. The formula is:

$$s = \frac{\mathrm{SQ}}{\mathrm{I}}.$$

The first step is to determine position of center of gravity of flange. As there are no cover-plates, this is taken directly from "Cambria" and is 1.64 inches.

The web is 36 inches; but in all girders where flange plates are used, the depth back to back of angles is $\frac{1}{4}$ or $\frac{1}{2}$ inch more than the depth of web, in order to allow for any variation in the depth of plate. In this case it will be taken as $36\frac{1}{4}$ inches back to back of angle.

$$\mathrm{Q} = 2 \times 4.36 \times 16.49 = 143.8$$

$$\begin{aligned} \mathrm{I} = 4 \times 15.39 &= 61.6 \\ 4 \times 4.36 \times \overline{16.49}^2 &= 4{,}740. \\ \tfrac{1}{12} \times \tfrac{3}{8} \times \overline{36}^3 &= 1{,}458. \\ &\ 6{,}259.6 \end{aligned}$$

$$\mathrm{S} = \frac{60{,}000 \times 143.8}{6{,}259.6} = 1{,}375$$

$$\text{Pitch} = \frac{5{,}060}{1{,}375} = 3.69 \text{ inches.}$$

Something less than this would be actually taken--probably $2\frac{3}{4}$ or 3 inches.

To determine the pitch at point 10 feet from end, we have to calculate the neutral axis of the flange as follows:

$$\begin{array}{lrcl}
\text{Angles } 2 \times & 4.36 \times 2.39 & = & 20.9 \\
& 10.5 \times .38 & = & 4.0 \\
\hline
& & & 24.9
\end{array}$$

$24.9 \div 19.22 = 1.3$ inches from back of cover-plate to neutral axis.

$$\begin{array}{rcl}
Q = 19.22 \times 17.58 & = & 338 \\
I = 4 \times 15.39 & = & 61.6 \\
2 \times 19.22 \times \overline{17.58}^2 & = & 11,870. \\
\frac{1}{12} \times \frac{3}{8} \times \overline{36}^3 & = & 1,458 \\
\hline
& & 13,389.6
\end{array}$$

$$S = \frac{30,000 \times 338}{13,390} = 757$$

$$\text{Pitch of rivets} = \frac{5,060}{757} = 6.68 \text{ inches.}$$

The maximum pitch is as stated 6 inches. At this point the actual pitch would be made somewhat less—say, $5\frac{1}{2}$ inches.

As a comparison with the foregoing results, it will be well to note

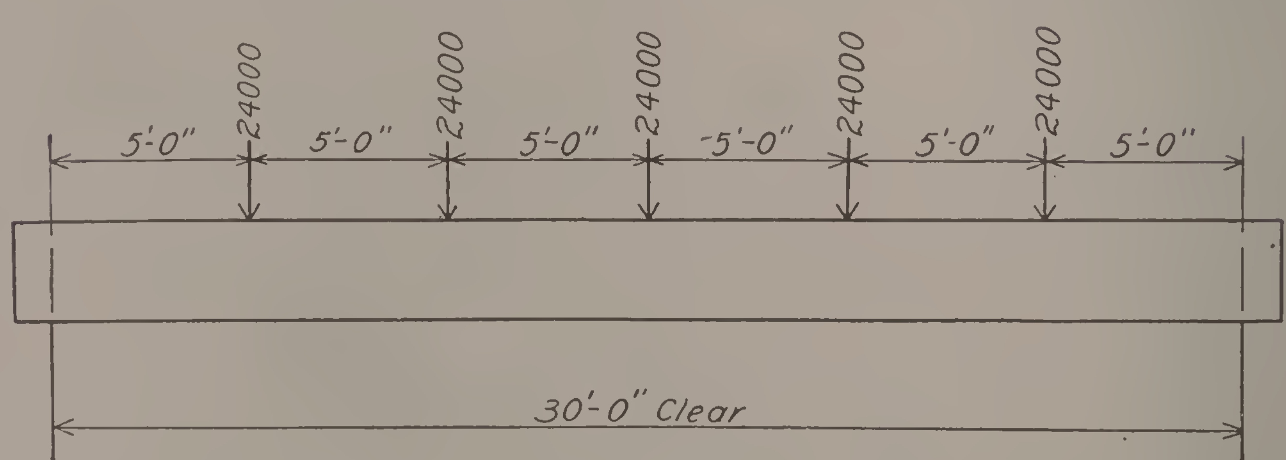


Fig. 255.

the pitch as determined by the approximate method, using the distance between centers of gravity of flanges. At the ends, we have

$$\frac{60,000}{33} = 1,820$$

$$\text{Pitch} = \frac{5,060}{1,820} = 2.78 \text{ inches.}$$

It will be seen that this approximate method gives some closer pitch than the more exact formula.

2. Given a girder 30 inches by $\frac{5}{16}$ inch, $30\frac{1}{4}$ inches back to back of

3 Girders Like This
Mark A1 A2 A3

Bill of Material for 3 Girders.

Item	No. of Pcs.	Kind	Size	Length Feet	Length Ins.	Work
	3	Web Pls.	$30'' \times \frac{3}{8}''$	28	0	
	12	Flge. Ls.	$5'' \times 5'' \times \frac{1}{2}''$	28	0	
	6	Flge. Pls.	$12'' \times \frac{1}{2}''$	28	0	
A	12	Stiffr. Ls.	$4'' \times 3'' \times \frac{3}{8}''$	2	$5\frac{1}{2}$	Fitted Top & Bottom
B	12	Stiffr. Ls.	$5'' \times 3'' \times \frac{3}{8}''$	2	$5\frac{1}{2}$	Fitted Top & Bottom
C	24	Fills.	$3'' \times \frac{1}{2}''$	1	8	
D	24	Stiffr. Ls.	$5'' \times 3\frac{1}{2}'' \times \frac{5}{16}''$	2	$5\frac{1}{2}$	Fitted Top & Bottom
E	24	Fills.	$3\frac{1}{2}'' \times \frac{1}{2}''$	1	8	
F	6	Bed Pls.	$12'' \times \frac{3}{4}''$	1	4	Countersunk

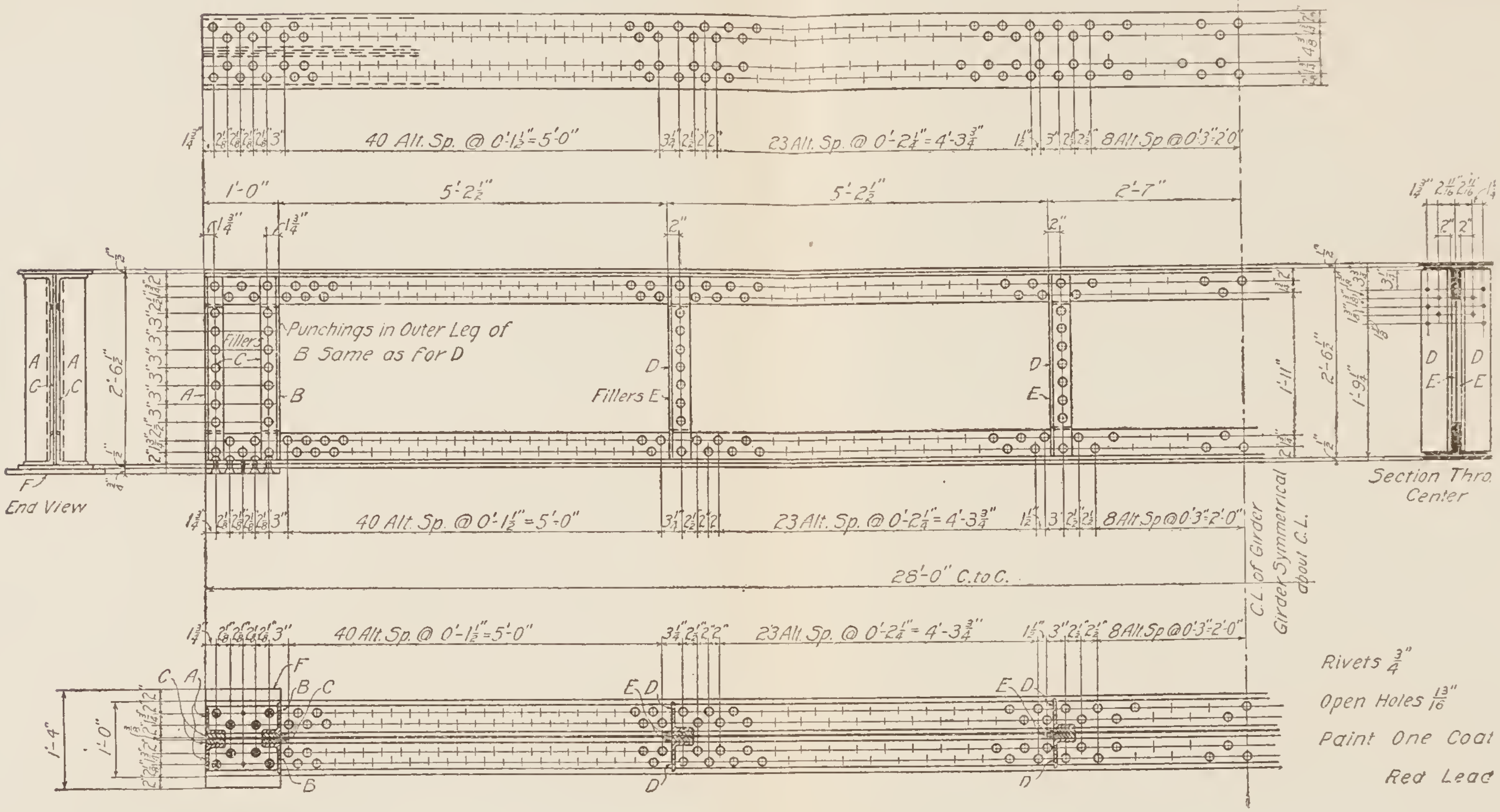


Fig. 256.

flange angles. The flange section is made up of two angles $4 \times 4 \times \frac{3}{8}$ inch. The end shear is 42,000 pounds. Determine the pitch of rivets by the approximate method.

3. Given a girder 42 feet long, loaded with a uniformly distributed load of 7,000 pounds per linear foot. If the web is 42 inches by $\frac{7}{16}$ inch, and the flange section at the end is made up of two angles $6 \times 6 \times \frac{1}{2}$ inch, and 1 plate $14 \times \frac{1}{2}$ inch, and distance back to back of angles is $42\frac{1}{4}$ inches, (*a*) determine the pitch of horizontal rivets through web; (*b*) determine the pitch of vertical rivets through flange plates. Give two solutions of (*a*) and (*b*), using the exact formula and the approximate method based on distance between centers of gravity of flanges.

Answers—(*a*) $1\frac{5}{8}$ inches by the approximate method.
$1\frac{15}{16}$ inches by the exact method.
(*b*) $3\frac{1}{4}$ inches by the approximate method.
$6\frac{3}{8}$ inches by the exact method.

Note that where pitch of vertical rivets through cover-plates is determined by the approximate method, they are simply assumed as alternating with the horizontal rivets. If there is only one line of horizontal rivets through flange angle and web, and one line of vertical rivets, then, by the approximate method, the vertical rivets through cover-plates would come centrally in the space between the horizontal rivets. If there are two lines of horizontal rivets, and one line of vertical, the vertical rivets would still alternate with the inner line of horizontal rivets, or center over the outer line of horizontal rivets. This would hold good so long as the spacing in this way did not exceed 6 inches, or sixteen times the thickness of plate. If this were the case, then the vertical rivets would be made to center over each line of horizontal rivets. The same practice as regards vertical rivets would be followed in case both horizontal and vertical legs had two lines of rivets. The formula for exact determination of rivet pitch shows that the above approximate methods are within the limits which would be determined if the exact method was used.

Shop Details of Girders. Fig. 256 is a shop detail of a simple plate girder of one web. It will be noted that the detail covers only one-half the girder. Where the girder is exactly symmetrical about the center line, it would be a waste of time to draw up both halves. In such cases it is sufficient to mark the center line and mark the draw-

ing so that it will be clear that the other half is the same. In some cases where there is only a slight difference, as at the ends between the two halves, it is still unnecessary to detail more than half the girder; in such cases a special detail of the end which is different should be added.

This girder rests on a brick wall at each end; and therefore the end stiffeners are placed over the outer edge of bearing plate, as shown. A wall rests on top of the girder, and the intermediate stiffeners are to support the flange when the main pier lines come down, and to stiffen the web for the concentrated beam loads.

A girder such as this would probably come into the drafting room for details with only such information as is given in Fig. 257.

In many cases, even the loading on the girder might not be given. In such case, it would have to be calculated from the general plans

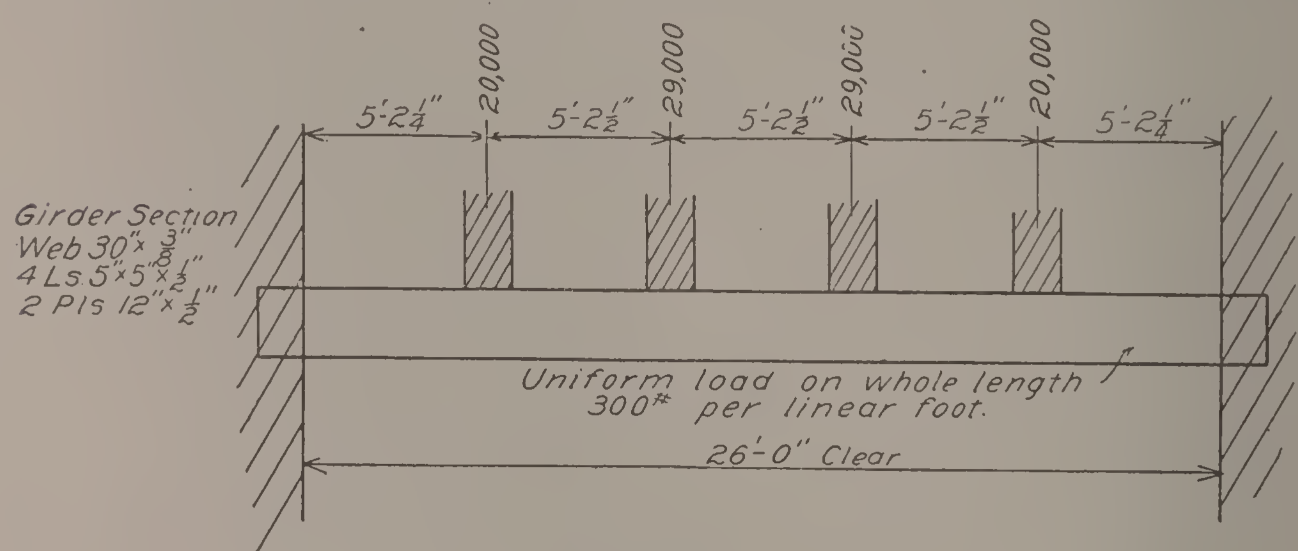


Fig. 257.

showing amount and distribution of floor and wall loads. If the loads had been uniformly distributed, details might have been made by determining the capacity of the girder, as noted below.

The first point to be determined is the size of the bearing plate. The reaction is 65,000 pounds; and, allowing a safe bearing on the stone template of 25 tons per square foot, this requires about 1.30 square feet. A plate 12 by 16 inches, therefore, will be sufficient. Applying the formula given on page 97 of Part II, the required thickness is found to be .26 inch; a steel plate ¾ inch thick is used here, although ½-inch plate might have been used.

The size of the bed-plate having been fixed, the spacing of all the stiffeners is the next thing to determine. The end ones are fixed at 12 inches back to back. As the piers come down on top of the

girder, it will be sufficient to use one stiffener in the center of each pier; if the pier was very heavy or over 3 feet, it would be well to use two under each pier. The measurements given in the diagram (Fig. 257), therefore, fix the other stiffeners. It is then necessary to look into the shear on the web to see if stiffeners are required on this account. Referring to the diagram (Fig. 246), it is found that for a $\frac{3}{8}$-inch web and 18 inches between edges of flange angles, the allowable shear per square inch of web is 6,800 pounds. The actual shear is $\frac{76{,}000}{30 \times \frac{3}{8}} = 6{,}750$ pounds, which is therefore entirely safe without stiffeners, as the shear just one side of the end is 11,000 pounds less.

Looking now at the horizontal rivet spacing, we find, at the end,

$$s = \frac{65{,}000}{28} = 2{,}320 = \text{approximate horizontal shear per inch.}$$

Some engineers use the distance between pitch lines of flange rivets, or, in case of double pitch lines, the mean between the two, instead of using distance between centers of gravity for determining the approximate shear. In this case the result would be:

$$s = \frac{65{,}000}{24.75} = 2{,}630 \text{ pounds.}$$

The bearing value is the least for these rivets, and may be taken at 5,060; the end pitch, therefore, is $\frac{5{,}060}{2{,}630} = 1.92$ inches.

It is always better to space a little under the calculated pitch; for this reason $1\frac{1}{2}$ inches was used.

The loads being concentrated, the shear is practically constant from the end to the first stiffener; and the only other point to consider is to space from each stiffener so as to conform to the standard gauge in the stiffener angle, and to keep this where previously fixed, leaving room from the back of angle to drive first rivet. The distance back to back being 5 feet $2\frac{1}{2}$ inches, and the standard gauge in one case 2 inches and in the other $1\frac{3}{4}$ inches, the distance center to center of pitch lines in stiffener is 5 feet $6\frac{1}{4}$ inches. It is well to leave not less than 1 inch, and better $1\frac{1}{4}$ inches, from the back of a stiffener to first rivet so that it can be easily driven; leaving $1\frac{1}{4}$ inches will just allow for 40 spaces at $1\frac{1}{2}$ inches.

The shear just to the right of the first stiffener from the end, is 25,000 pounds; therefore, $s = \dfrac{25,000}{24.75} = 1,010$ pounds.

The direct shearing force from the pier load is $\dfrac{30,000}{24} = 1,250$ pounds per inch.

If we assume a pitch of 3 inches, this brings 3,750 pounds on each rivet, and the diagram of stress would be as illustrated in Fig. 252, the resultant stress being about 4,850 pounds. A pitch of 3 inches could therefore have been used and need not have been continued much beyond the pier lines. In order to keep the pitch constant, however, and be somewhat under the required pitch, $2\frac{1}{4}$ inches was used. Similarly, the pitch in center way is made 3 inches, although somewhat larger pitch might have been used.

The actual required pitch through flange plates would be found much less than shown, since there are four lines of rivets instead of two as is commonly the case in girders of this length. In order to simplify the shop work, however, they are detailed the same spacing. It is well to note that in such cases the rivet through flange plate on the gauge line nearest to the vertical leg of flange angle, comes opposite the vertical rivet in flange line farthest from the horizontal leg. This is to give all possible room for riveting, and also because it distributes the rivets more uniformly.

The bottom flange spacing is made the same as top, and differs only in having the rivets through bearing plates countersunk, with open holes for anchor bolts.

The bill of material should be clear after explanation given in Part III for bills of columns.

Fig. 258 shows the detail of a two-web girder. This girder carries a wall on a street front, and is one of a continuous line of several girders. The right-hand end is at the corner of the building; and the open holes shown are for connection of a girder on the other street front. The girder rested on steel columns, and the arrangement of the line members of the columns determined the spacing and arrangement of the end stiffeners on the girder.

The column section coming under right-hand end is shown by Fig. 259. The stiffeners at the extreme left end are simply for connection to similar stiffeners on the end of the girder coming against

BILL OF MATERIAL FOR ONE GIRDER

ITEM	No of PCS	KIND	SIZE	LENGTH FEET	LENGTH IN	WORK
	2	Web Pl's	$30'' \times \frac{1}{2}''$	21	$5\frac{1}{4}$	
	1	Flge Pl	$18'' \times \frac{1}{2}$	21	$5\frac{1}{4}$	
	1	" "	$18'' \times \frac{1}{2}''$	15	$1\frac{1}{4}$	
	1	" "	$19'' \times \frac{1}{2}$	21	$5\frac{1}{4}$	Countersunk
	1	" "	$19 \times \frac{1}{2}$	15	$1\frac{1}{4}$	
	4	Flge Ls	$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	21	$5\frac{1}{4}$	Countersunk
A	12	Angles	$3'' \times 3'' \times \frac{3}{8}''$	2	$5\frac{1}{4}$	Fitted Top & Bottom
B	2	Fillers	$8\frac{3}{8}'' \times \frac{1}{2}''$	1	$10\frac{1}{2}$	
C	2	Angles	$3'' \times 2\frac{1}{2}'' \times \frac{5}{16}$	2	$5\frac{1}{4}$	Fitted Top & Bottom
D	8	Fillers	$3'' \times \frac{1}{2}''$	1	$10\frac{1}{2}$	
E	4	Angles	$4'' \times 3\frac{1}{2}' \times \frac{3}{8}''$	2	$5\frac{1}{4}$	Fitted Top & Bottom
F	4	Fillers	$4' \times \frac{1}{2}''$	1	$10\frac{1}{2}$	

ONE GIRDER LIKE THIS MARK NO. 7

All Rivets $\frac{3}{4}''$
All Open Holes $\frac{13}{16}''$
Paint One Coat Graphite

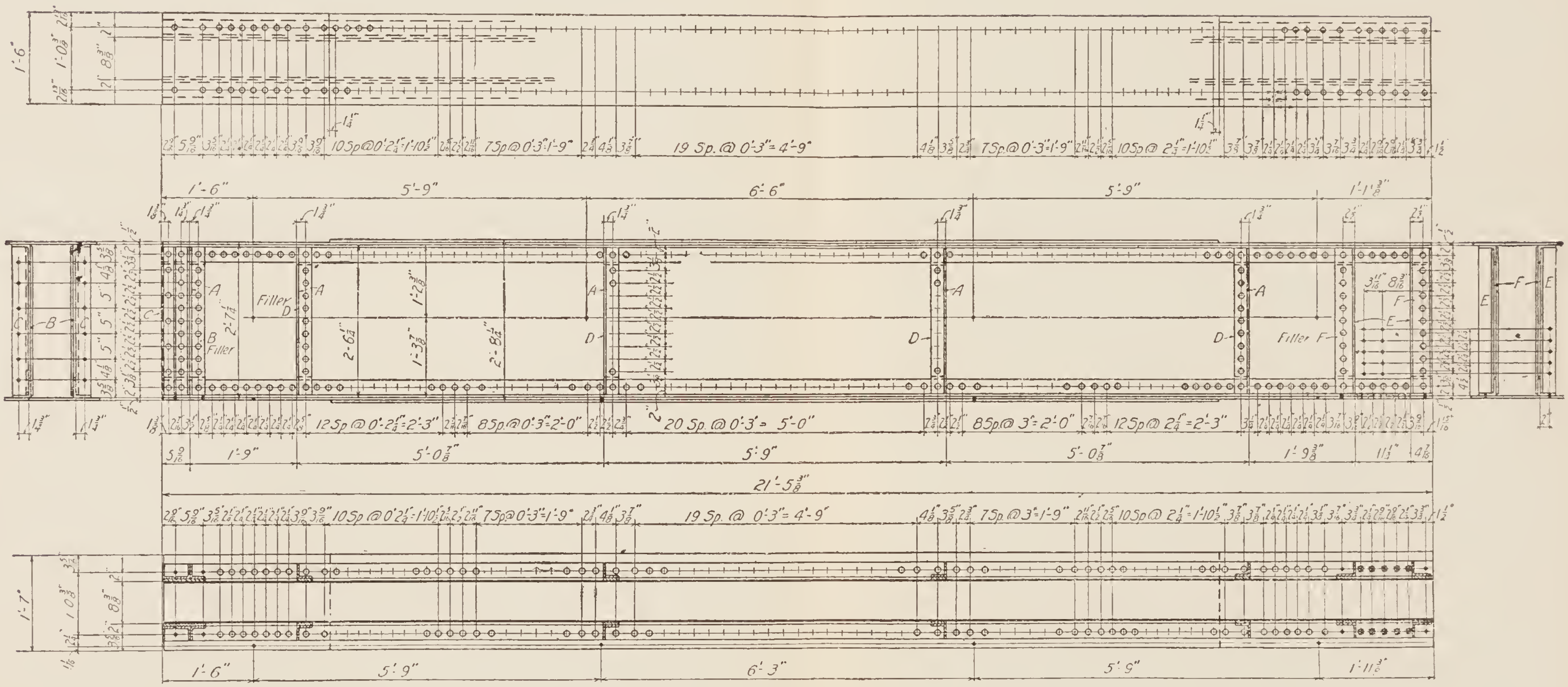


Fig. 258.

this one. The intermediate stiffeners are for support of flange under centers of brick piers.

The bottom plates were made 1 inch larger than the top plates for the purpose of securing the ornamental fascia.

In the calculation of the rivets of a two-web girder, the shear is assumed to be divided equally on the two webs; and therefore each line is calculated as before described, except that the shear used is one-half the total. It should be noted, also, in such cases, that the rivets are in single shear.

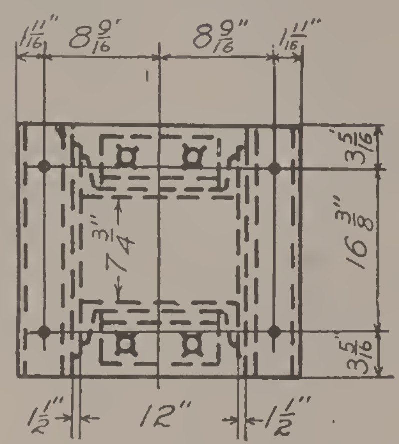


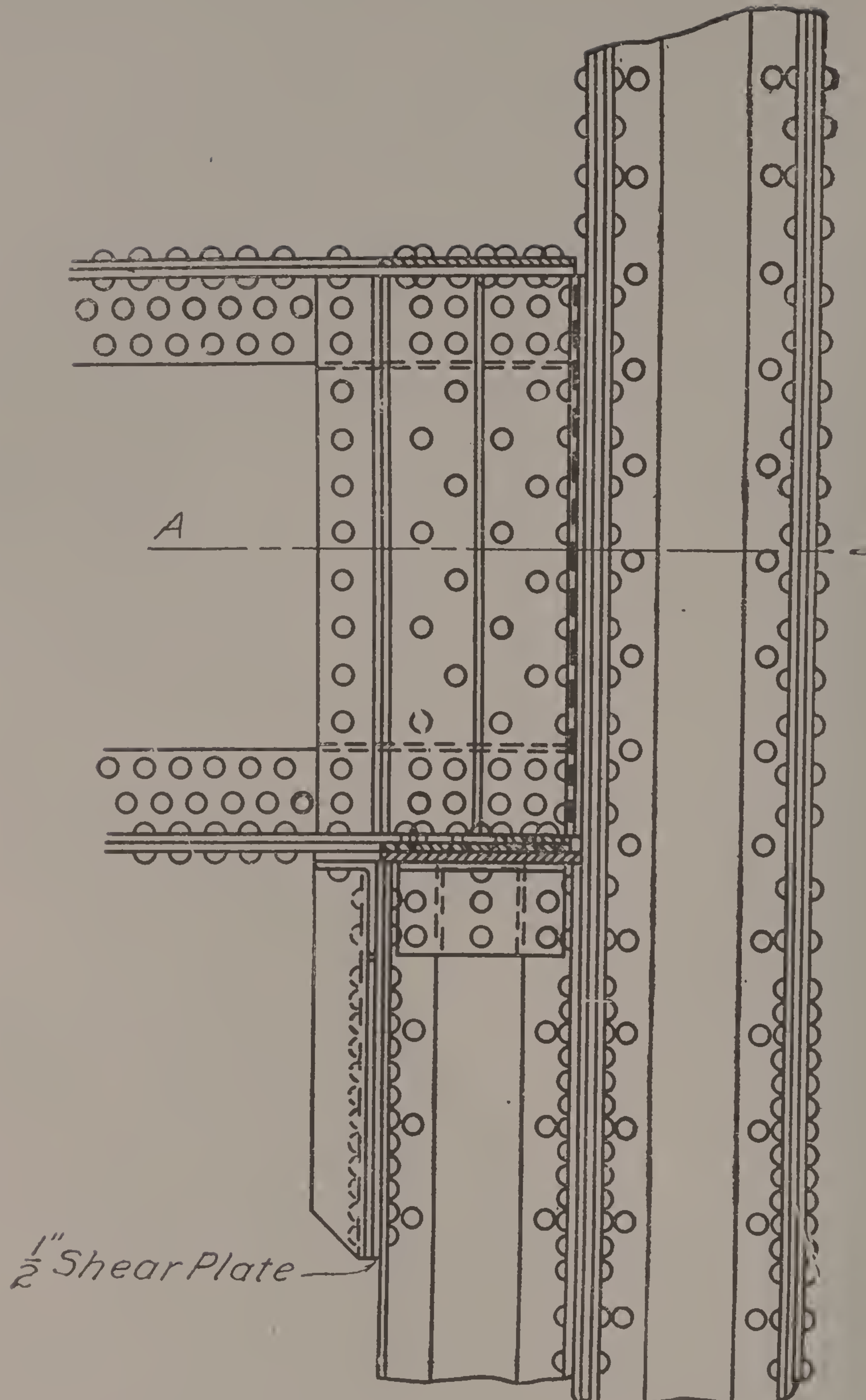
Fig. 259.

One plate must, of course, be made the full length of the girder. The length of the other plate is determined as previously described, and a length added at each end sufficient to get rivets equal to one-third the capacity of the plate. In this case, the net area of the plate being about 8.2 inches, the capacity is 123,000 pounds; and the required number of rivets in single shear is 10, or 5 in each line.

It should be noted that in two-web girders it is possible to have flange angles only on the outside of the web, as the only way inside angles could be riveted would be by working a man from the end in between the webs. This is ordinarily impossible on account of the small space between, and would always be too expensive to justify such designs.

Fig. 260 gives the detail of a three-web girder. This girder is in the street front of a modern steel-framed office building, and spans the large store fronts which are made possible by stopping one of the main lines of columns on top of this girder. The girder rests on columns at each end, as shown by Fig. 261, and is symmetrical with respect to the center line. It will be noted from Fig. 261 that the column carrying the end of this girder is practically made up of two columns riveted together through their flanges. This construction permits the heavy girder to get a bearing directly over the column shaft, and continues in a direct line the axis of the column section above and the portion of this column carrying these upper sections. This girder also carries the floor beams, which frame into the bottom flange as illustrated in Fig. 262.

There are some points of a practical nature which should be



Section A-A.

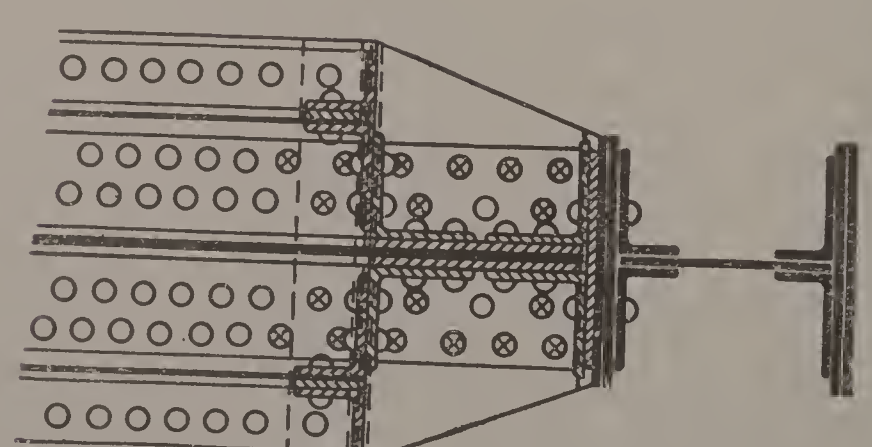


Fig. 261.

2 GIRDERS LIKE THIS
MARK 3RD FLOOR B & C

BILL OF MATRIAL FOR 2 GIRDERS

ITEM	No of PC'S	KIND	SIZE	LENGTH FEET	LENGTH IN.	WORK
	2	Web Pl's	$36'' \times \frac{1}{2}''$	25	$2\frac{1}{4}$	
	4	" "	$36'' \times \frac{3}{8}''$	23	2	
	8	Angles	$6'' \times 6'' \times \frac{1}{2}''$	25	$2\frac{1}{4}$	
	8	" "	$6'' \times 4'' \times \frac{1}{2}''$	23	2	Countersunk
	4	Flge Pl's	$24'' \times \frac{9}{16}''$	25	$2\frac{1}{4}$	" Bevelled
	4	" "	$24'' \times \frac{9}{16}''$	14	$11\frac{3}{8}$	"
	4	" "	$24'' \times \frac{9}{16}''$	10	$6\frac{3}{8}$	"
	4	" "	$24'' \times \frac{9}{16}''$	6	$1\frac{1}{8}$	"
A	8	Stiff'r. Ls	$6'' \times 6'' \times \frac{5}{8}''$	2	$11\frac{1}{2}$	Fitted Top and Bottom
B	16	" "	$6'' \times 6'' \times \frac{1}{2}''$	2	$11\frac{1}{2}$	" " " "
C	8	" "	$4'' \times 4'' \times \frac{1}{2}''$	2	$11\frac{1}{2}$	Sheared to $4'' \times 3\frac{1}{2} \times \frac{1}{2}''$
D	8	" "	$6'' \times 4'' \times \frac{1}{2}''$	2	$11\frac{1}{2}$	Fitted Top and Bottom

BILL OF MATERIAL CONTINUED

E	8	Fill's	[illegible]	2	0	
F	8	"	[illegible]	2	0	
G	8	Stiff'r Ls	$6 \times 3\frac{1}{2} \times \frac{1}{2}$	2	$11\frac{1}{2}$	Fitted Top
H	8	" "	$6 \times 6 \times \frac{1}{2}$	2	$11\frac{1}{2}$	" "
J	8	Fill's	$6 \times \frac{1}{2}$	2	0	
K	4	"	$13\frac{1}{2}'' \times \frac{1}{2}''$	2	0	
L	4	Angles	$6 \times 6 \times \frac{1}{2}$	0	$7\frac{5}{8}$	Bevelled
M	4	Fill's	$5\frac{1}{2} \times \frac{1}{2}$	0	$4\frac{5}{8}$	
N	4	Angles	$6 \times 6 \times \frac{1}{2}$	0	$7\frac{5}{8}$	Bevelled
O	4	Fill's	$5\frac{1}{2} \times \frac{1}{2}$	0	$4\frac{5}{8}$	
P	2	Angles	$6'' \times 6 \times \frac{1}{2}$	0	$7\frac{5}{8}$	Bevelled
Q	2	Fill's	$5\frac{1}{2} \times \frac{1}{2}$	0	[illegible]	
R	4	Pl's	$12\frac{1}{2} \times \frac{3}{8}$	3	[illegible]	Shop bolted in place

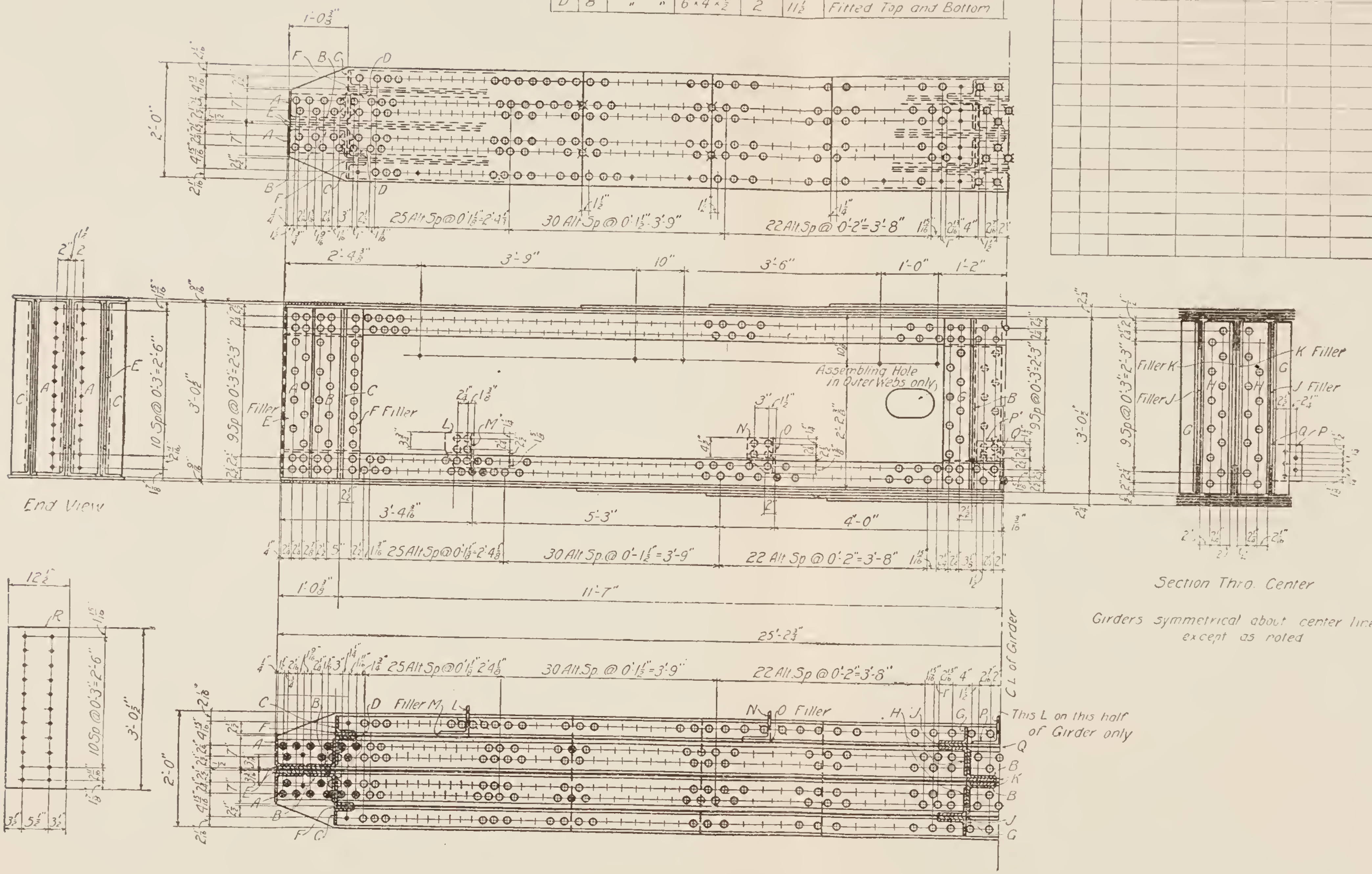


Fig. 260.

noted on this detail. In a heavy girder of three webs, there are practical difficulties to be met with in riveting. These must be considered and provided for in making the details.

The steps in assembling this girder would be:

(1) Rivet up the central portion, consisting of web and four angles.

(2) Rivet the top and bottom flange plates to this central portion of the girder.

(3) Rivet up each side portion, consisting of web-plate and two angles.

(4) Rivet each side section to the flange plates, which have previously been riveted to the central portion.

It will be noted that the position of stiffeners is somewhat different

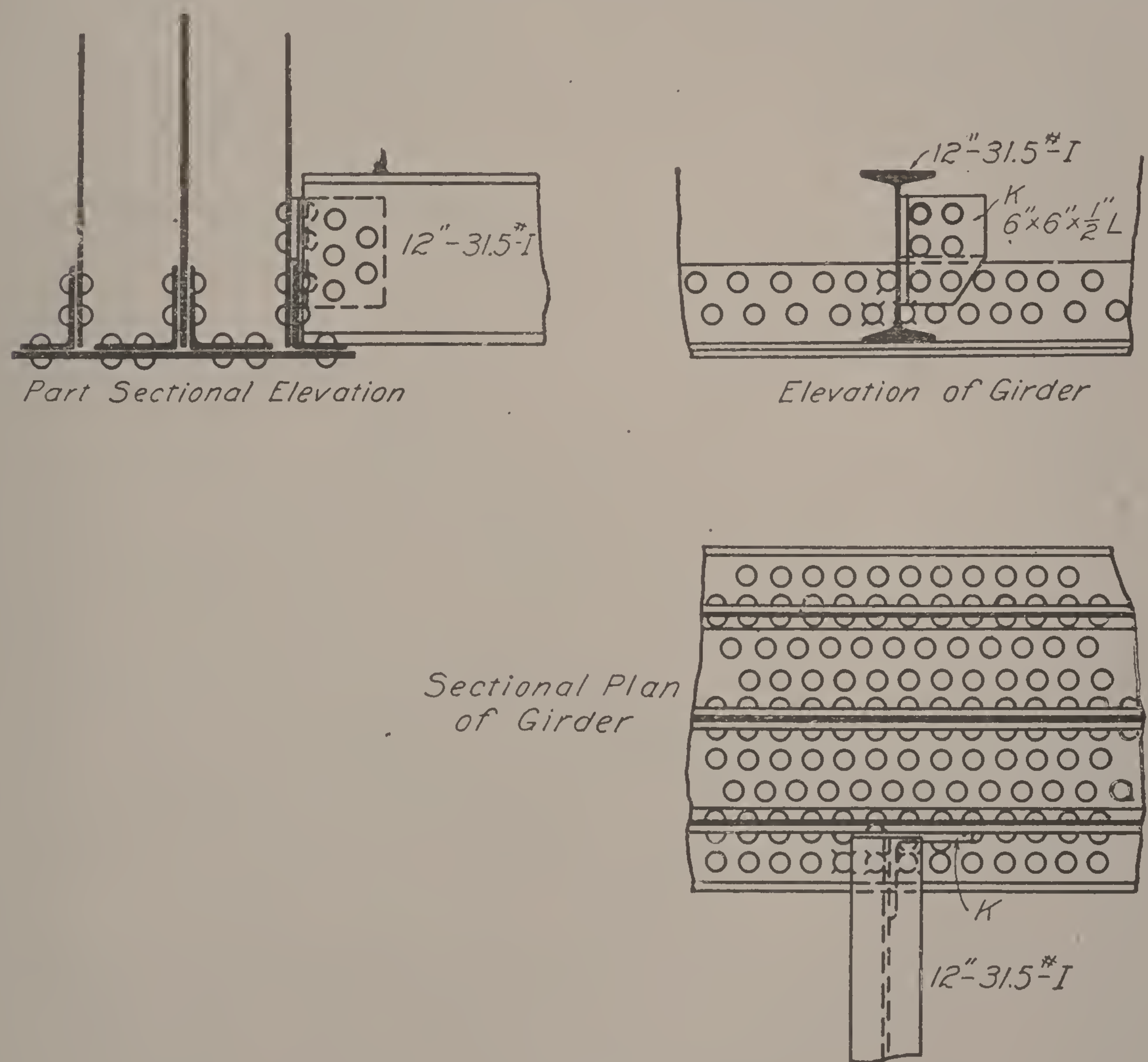


Fig. 262.

from what has previously been described. The stiffeners A and B at the end are placed so as to come down directly over the line members of the column below. The stiffeners C and D are placed so as to

come over the shear plate on the column. B and D are also so placed that they can be riveted together and thus form a plate stiffener between the three webs. To rivet up B and D, it is necessary to rivet them together first; then rivet D to the side webs and angles C before these side webs are assembled with the central web. After the side webs are assembled, B can be riveted to the central web.

The stiffeners at the center of the girder are arranged to come under the line members of the column resting on the top flange of girder, and also to serve as plate stiffeners for the webs.

The method of procedure for riveting up these stiffeners is somewhat different from that used in case of the end ones. In this case, B and H would be riveted together, and then B riveted to the central web before the side webs are assembled.

In order to rivet H and G to the side webs, it is necessary to provide a hand hole in each side web as shown, so that these rivets can be held on the back side while being driven up after the side webs are assembled.

In three-web girders the distribution of the shear over the three webs depends to a considerable degree on the way in which the loads are applied. It is generally considered that the center web takes the larger proportion, sometimes as much as $\frac{5}{8}$, and the side webs take the remainder equally. These webs should always be stiffened so as to distribute all loads as much as possible over all three webs.

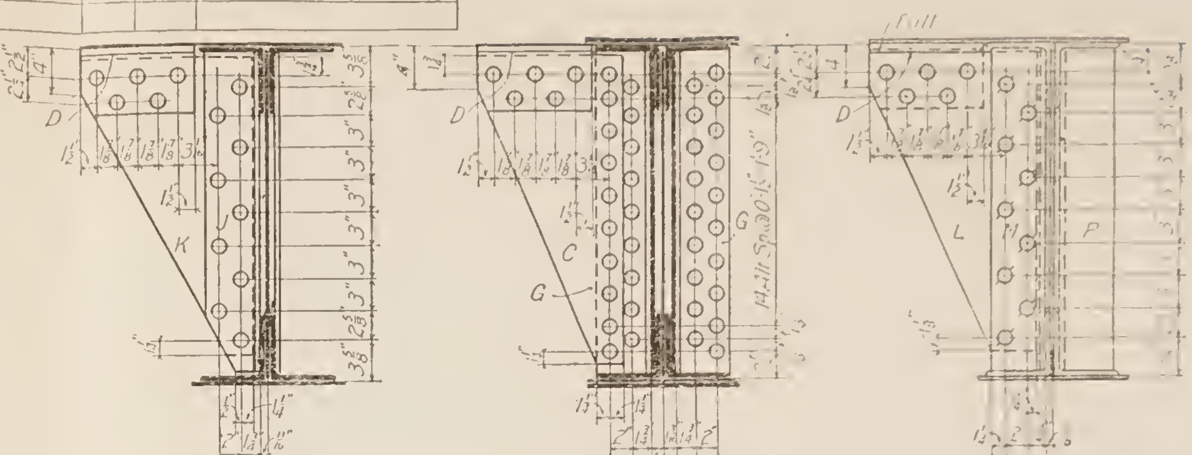
The designer, in choosing his sections, will necessarily make an assumption as regards this distribution; and this should be indicated on the diagram. Practically the pitch in all three webs and flange angles would be made the same, this being determined so as to provide for the maximum shear according to the assumption as regards distribution. The actual number of rivets may vary in the different portions, because of angles being used which may allow of only one line of rivets, as in the case shown in Fig. 260.

The detail of connection of floor beams to girder is made special because of the awkward relation of beams to girder flanges, which relation could not be changed; only a single angle could be used for the connection if this was to be riveted on, and this had to be shipped riveted to girder rather than beam. It would have been possible to have a double-angle connection by using an intermediate plate and two side plates; but this would have added to the expense of erection,

BILL OF MATERIAL FOR ONE GIRDER

ITEM	NO OF PCS	KIND	SIZE	LENGTH FEET	LENGTH IN	WORK
	1	Web Pl	30" × 3/8"	13	2	
	2	" "	28" × 1/2"	5	5 1/2	Countersunk
	4	Flge Ls	6 × 6 × 1/2	13	2	
	1	" Pl	24" × 5/16	13	2	
	1	" "	14" × 5/16	11	11 1/4	
	2	" "	14" × 7/16	5	5 1/2	
A	1	Stiff'r L	5 × 3 1/2 × 1/2	2	5 1/2"	Fitted Top & Bottom
B	1	" "	5 × 3 1/2 × 1/2"	2	5 1/2	" " "
C	3	Plates	13 5/16 × 3/8"	2	5	Bevelled
D	10	Angles	6" × 3 1/2" × 1/2"	0	10 1/2	"
E	4	Stiff'r Ls	5" × 3" × 3/8"	2	5 1/2	Fitted Top & Bottom
F	2	" "	5" × 3 1/2" × 1/2"	2	5 1/2	" " "
G	8	" "	5" × 5" × 1/2"	2	5 1/2	" " "
H	4	Plates	4 3/4" × 3/4"	2	5 1/2	" " "
J	4	Angles	5" × 3 1/2" × 3/8"	2	5	Bevelled
K	2	Plates	15 5/16" × 3/8"	2	2 3/8	"
L	1	"	15 5/16" × 3/8"	2	2 3/8	"
M	1	Stiff'r L	5 × 3 1/2" × 1/2"	2	5 1/2	Fitted Top & Bottom
N	2	Fillers	18 × 1/2	3	11 1/2	
O	2	"	4 7/8" × 1/2"	0	8	Ship bolted in place
P	1	Stiff'r. L	5" × 3 1/2" × 1/2"	2	5 1/2	Fitted Top & Bottom

All Rivets 3/4"
All Open Holes 13/16"
Paint One Coat Graphite



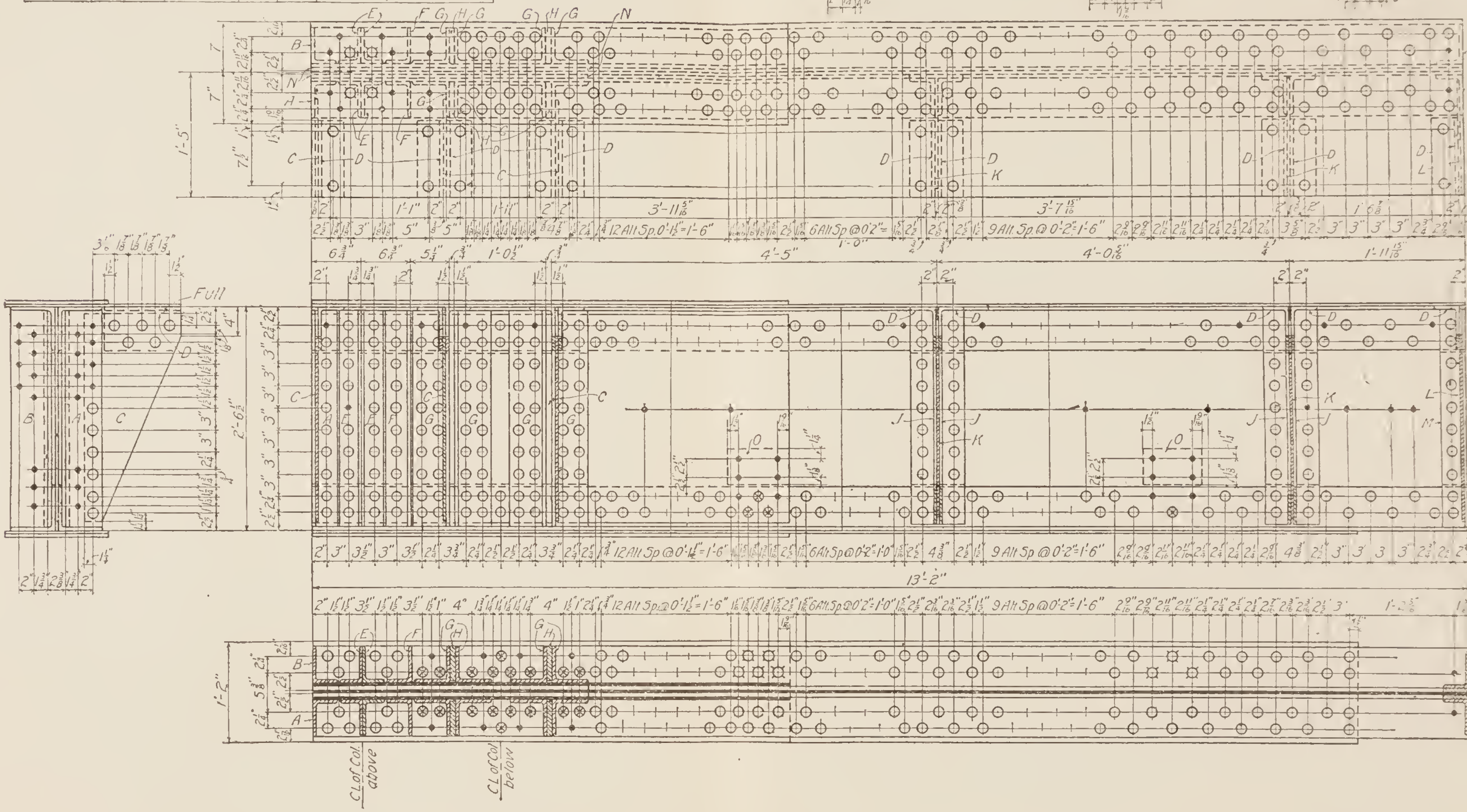


Fig. 263.

and sufficient rivets for the reaction were obtained by the single angle.

It will be noted that some rivets near these connections are shown flattened in the bottom flange to clear the flange of beams; also, in the elevation, some rivets are shown countersunk to clear the angle connection. Rivets are also shown countersunk where the cover-plates are left off, because there is not room to extend the plate beyond the last rivet without interfering with the next rivet. All such cases of countersinking or flattening rivets to avoid stiffeners or ends of flange plates, are to be avoided wherever possible, as they are objectionable and expensive. They can generally be avoided by changing the rivet spacing somewhat at such points. In the case shown in Fig. 260, the girder is such a heavy one, and the rivet spacing so close, that it was better to countersink rather than have the wide spacing otherwise necessary.

The end view shows open holes for riveting angles to the main column angles as shown in Fig. 261. This practice is objectionable for light girders, as previously noted in Part II, and where it is possible to properly brace the girder and column connection in any other way. In the case of a heavy girder such as this, where the deflection would be slight, it is not so objectionable, especially if these rivets are not driven until after the columns are carried up and the dead weight of construction is put upon the girder.

The bill of material should be carefully followed through as illustrating points previously mentioned.

Fig. 263 shows a single web-plate girder which carries the wall section over an entrance doorway, and also a column line on its cantilever end.

The center lines of the supporting column and of the column above, are shown on the plan of bottom flange. Fig. 264 shows the girder in its relation to the stonework, and the method of securing same to the girder.

The stiffeners G are arranged to come directly over the line members, and the shear angles on column below. The stiffeners A, E, and F are similarly arranged with respect to the column above carried on the end of the girder. It will be noted that this girder is not symmetrical about its center line, and therefore the detail of the whole girder is shown. It should be noted also that the concentration of loading at one end makes it necessary to increase the web greatly to

provide for the shear. For this reason a $\frac{1}{2}$-inch plate is riveted on each side over the flange angles and carried to a point beyond the cen-

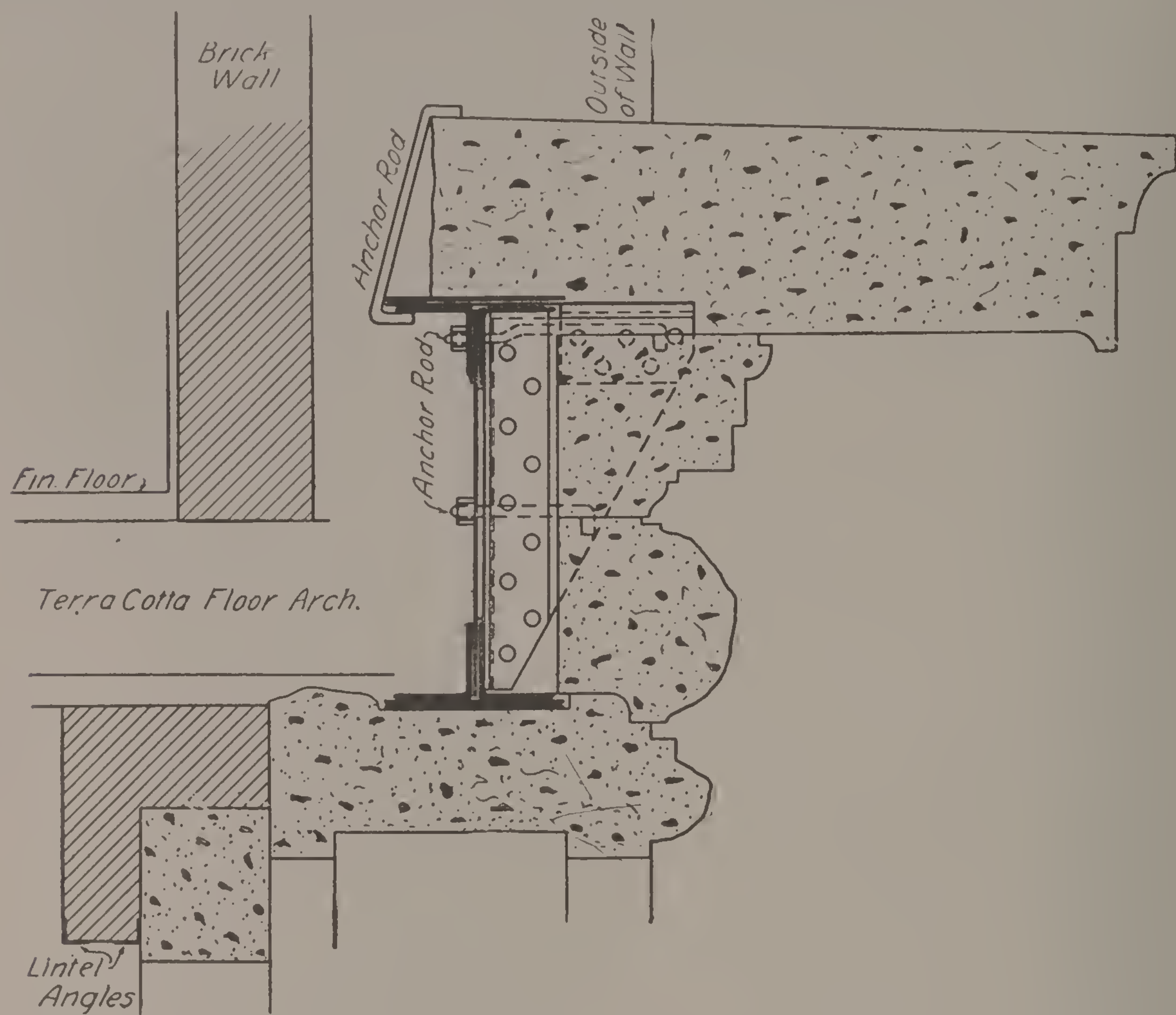


Fig. 264.

ter of column bearing where the area of the web alone is sufficient for the shear. This end being the point of maximum moment, also, is the reason for the increased flange area here.

Floor beams frame to this girder in the same relation as in the case of the three-web girder shown in Fig. 260; but as this is only a single web, the connection angles can be riveted to the beam. As the beam must be cut to clear the bottom flange angle, this necessitates a filler between the web and the connection angles on beam.

Note that where brackets or similar riveted members occur on a girder, it is better to give a separate section for the details of riveting of these members. The end view, and sections A, B, C, and D, show the details for these brackets supporting the stonework, and show the various details necessary to conform to the position and spacing of stiffeners on the girder.

In a girder loaded as this is, there should be sufficient area in each set of stiffeners coming under the column above and over the

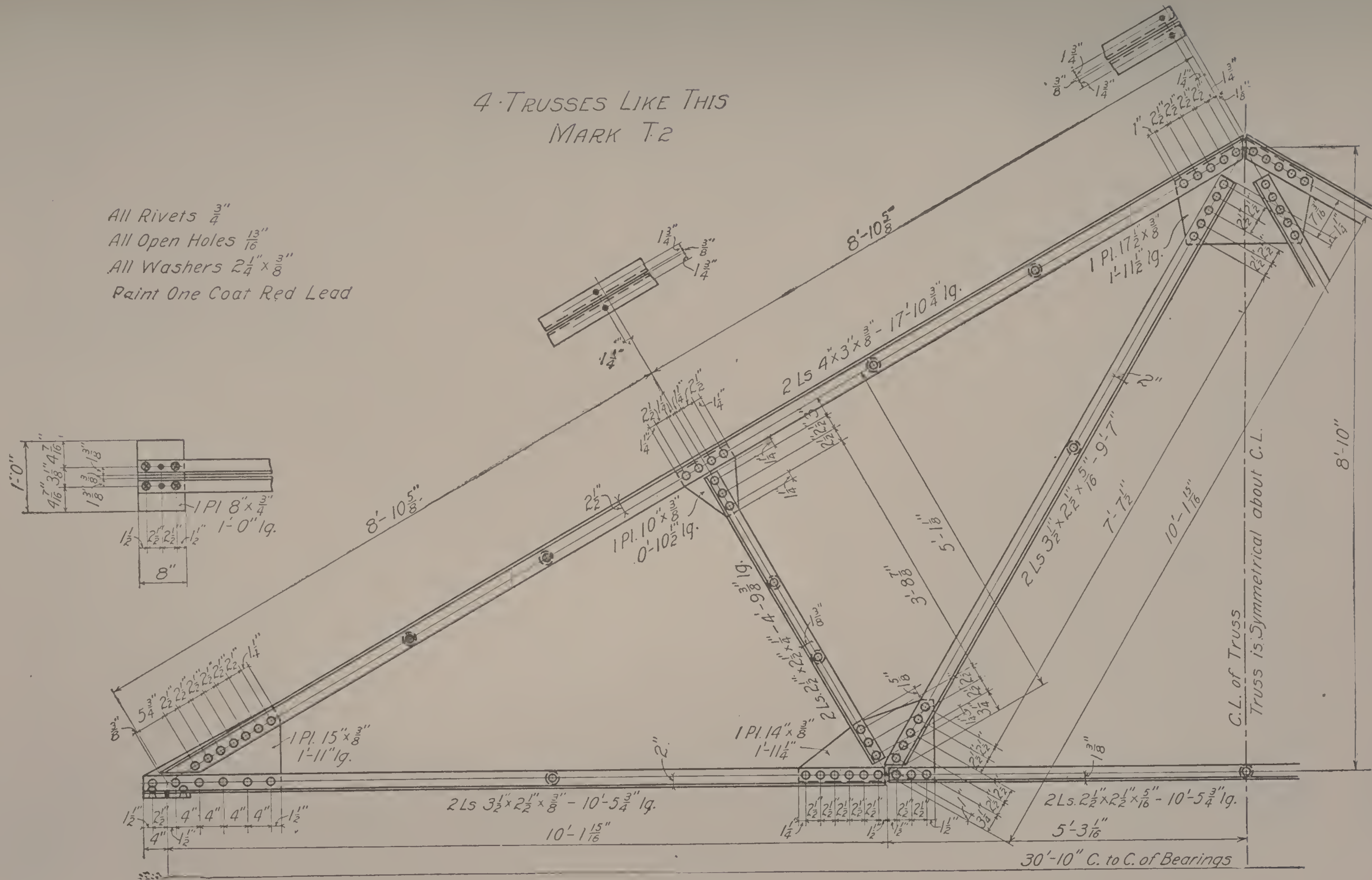

4 · Trusses Like This
Mark T2
All Rivets 3/4"
All Open Holes 13/16"
All Washers 2 1/4" × 3/8"
Paint One Coat Red Lead
2 Ls 4"×3"×3/8" - 17'-10 3/4" lg.
2 Ls 3 1/2"×2 1/2"×3/8" - 10'-5 3/4" lg.
2 Ls. 2 1/2"×2 1/2"×5/16" - 10'-5 3/4" lg.
2 Ls 3 1/2"×2 1/2"×5/16" - 9'-7"
2 Ls. 2 1/2"×2 1/2"×1/4" - 4'-9 3/8" lg.
1 Pl. 15"×3/8" 1'-11" lg.
1 Pl. 10"×3/8" 0'-10 1/2" lg.
1 Pl. 14"×3/8" 1'-11 1/4"
1 Pl. 17 1/2"×3/8" 1'-11 1/2" lg.
1 Pl 8"×3/4" 1'-0" lg.
8'-10 5/8"
8'-10"
10'-1 15/16"
7'-7 1/2"
5'-1 1/8"
3'-8 7/8"
5'-3 1/16"
30'-10" C. to C. of Bearings
C.L. of Truss
Truss is Symmetrical about C.L.

Fig. 265.

supporting column, to provide for the shear; and these stiffeners should be fitted to top and bottom flanges.

PROBLEMS

Make a complete shop detail, at a scale of $\frac{3}{4}$ inch to 1 foot, of a single-web plate girder 30 feet long clear span, resting on a brick wall at each end and carrying a load of 60 tons distributed as shown in Fig. 255. The web-plate is 30 inches by $\frac{3}{8}$ inch; both flanges have the same section, and each is made up of two angles 5 x $3\frac{1}{2}$ x $\frac{1}{2}$ inch (long leg horizontal), and two cover-plates 12 inches by $\frac{5}{16}$ inch. A 15-inch 42-pound beam frames to the girder on each side in the position indicated by loads. The top of the beams is $1\frac{1}{2}$ inches below the back of the flange angles. The beams are to rest on suitable shelf angles, with shear angles beneath, and have side connection angles riveted through web of girder to brace them laterally. Determine proper number of rivets and character of these connections. Determine number and spacing of stiffeners required. Use in addition stiffeners just one side of each beam connection.

Standards in Detailing Trusses. Figs. 265, 266, and 267 show details of various types of trusses. The same remarks made previously for girders apply to trusses wherever they are symmetrical about the center line.

Fig. 277 shows a strain sheet of the truss detailed in Fig. 266. This is the form in which the information is generally given to the draftsman for detailing. At other times the information may be given only by the general drawings, in which case the loads and measurements would have to be obtained from them.

It will be noted that the same general method of detailing and dimensioning is followed in all cases. The strain lines are laid out first; these lines should always intersect at the panel points; and the strain lines of the members over a point of support should intersect over the center of bearing. The strain lines should be theoretically the center of gravity lines of the members; it is more common practice, however, to use the pitch lines of the angles as the strain lines, as these lines do not vary materially from the center of gravity lines, and much confusion is thus saved. In heavy trusses, however, where the chords are made up of side plates and angles, the strain lines for the chords should be the center of gravity lines, as the difference between these lines and the pitch line of the angle would be considerable.

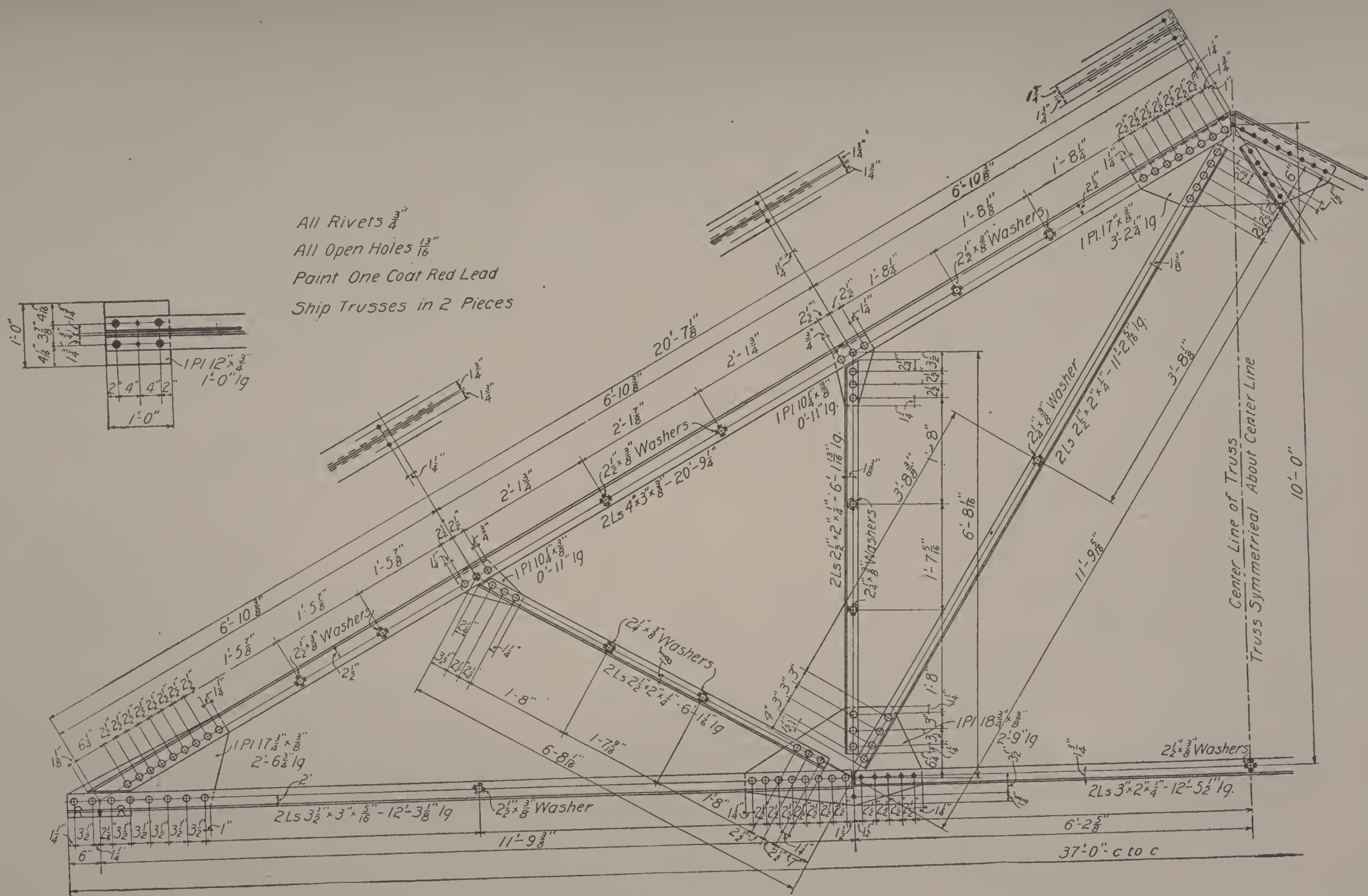

All Rivets 3/4"
All Open Holes 13/16"
Paint One Coat Red Lead
Ship Trusses in 2 Pieces
1 Pl 12"×3/4" 1'-0" lg
1 Pl 17 3/4"×3/8" 2'-6 3/4" lg
1 Pl 10 1/4"×3/8" 0'-11" lg
1 Pl 17"×3/8" 3'-2 1/4" lg
1 Pl 18 3/4"×3/8" 2'-9" lg
2Ls 4"×3"×3/8"-20'-9 1/4"
2Ls 3 1/2"×3"×5/16"-12'-3 1/8" lg
2Ls 3"×2"×1/4"-12'-5 1/2" lg
2Ls 2 1/2"×2"×1/4"-6'-1 1/16" lg
2Ls 2 1/2"×2"×1/4"-6'-1 13/16" lg
2Ls 2 1/2"×2"×1/4"-11'-2 5/16" lg
2 1/2"×3/8" Washers
2 1/4"×3/8" Washer
2 1/2"×3/8" Washer
20'-7 1/8"
11'-9 3/8"
6'-2 5/8"
10'-0"
37'-0" c to c
Center Line of Truss
Truss Symmetrical About Center Line

Fig. 266.

Many times the position of one or more panel points will be fixed by some features of construction such as a monitor or a hanger for shafting, or rod for balcony, etc., as illustrated by Figs. 267 and 280. Wherever such concentrated loads are fixed, there should be a panel point, if possible, as otherwise the chord must be materially increased to provide for the bending strains produced by the load acting between panel points. The panel points being fixed, and the strain lines drawn, the lines showing the size and shape of each member are drawn.

Completeness of Measurements. In dimensioning a detail the draftsman should bear in mind all the steps he has to take to fully lay out and fix all the members and connections, and should remember that information must be given to enable the templet maker to go through the same operations.

1. There should be measurements center to center of each panel point along each member. These are calculated, never scaled.

2. There should be a line of measurements along each member from panel point to panel point, fixing each rivet or hole with respect to this panel point.

3. There should be a measurement center to center of the end panel points along the top and bottom chords and the vertical or inclined end members.

4. There should be over-all measurements of the above members.

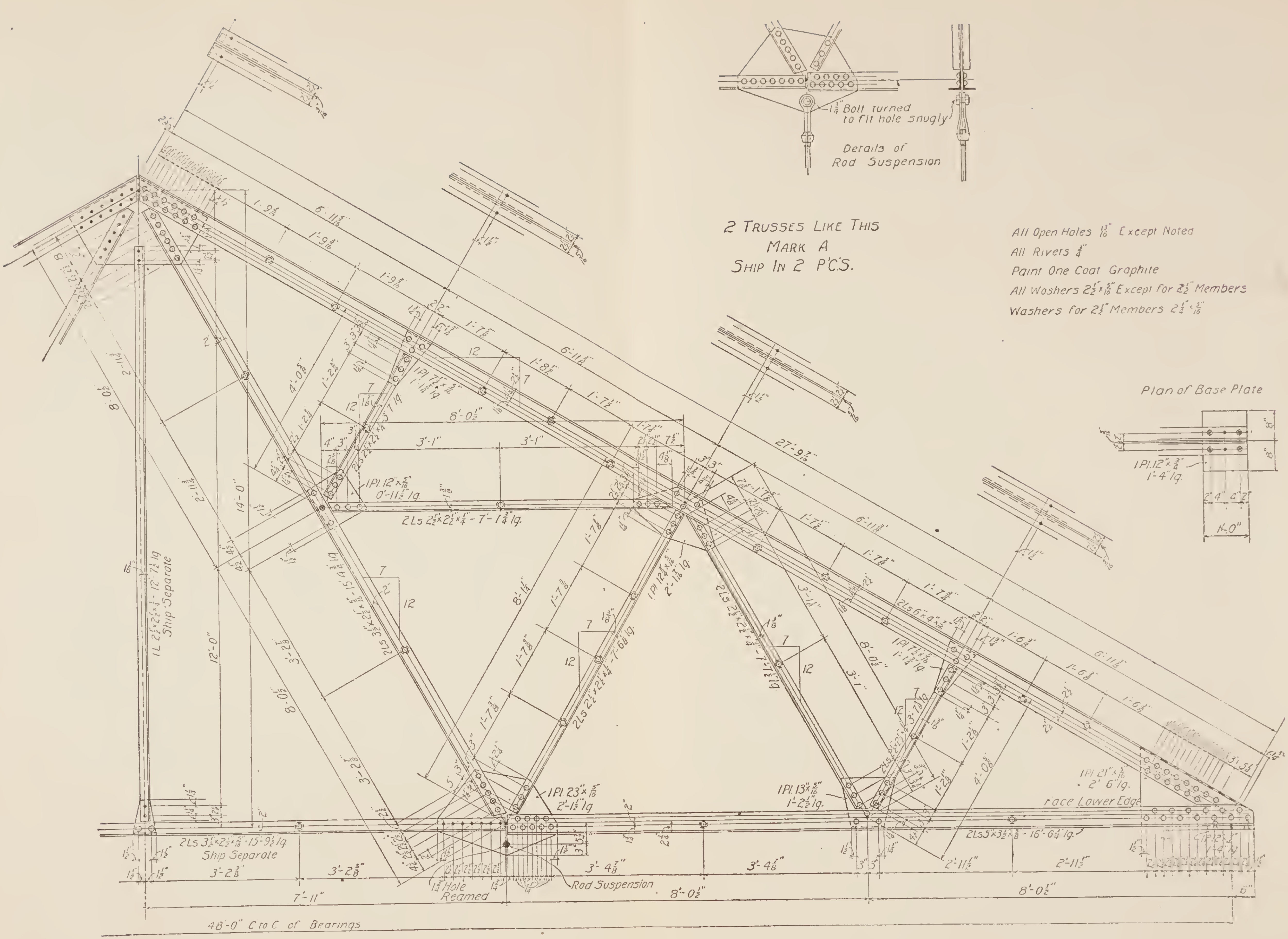
5. There should be a measurement from the end of each piece to the first rivet or hole, and each piece should have its size and over-all length specified.

6. Each sloping member should have its slope indicated by a triangle of which one side is 12 inches and the other side inches and sixteenths.

7. Each piece should preferably be given a shop mark, to facilitate assembling.

To fix the measurements noted under (2), it is often necessary to make a full-sized or large-scale layout drawn very accurately so as to be able to scale closely the distance from panel point to first rivet, and to be sure of plenty of clearance and yet have the members fit closely.

After the first hole is fixed, the others are spaced $2\frac{1}{2}$ or 3 inches apart for the gusset connections. The number of rivets is of course



2 TRUSSES LIKE THIS
MARK A
SHIP IN 2 P'C'S.
All Open Holes $\frac{13}{16}$" Except Noted
All Rivets $\frac{3}{4}$"
Paint One Coat Graphite
All Washers $2\frac{1}{2}$"×$\frac{5}{16}$" Except for $2\frac{1}{2}$" Members
Washers for $2\frac{1}{2}$" Members $2\frac{1}{4}$"×$\frac{5}{16}$"
$1\frac{3}{4}$" Bolt turned to fit hole snugly
Details of Rod Suspension
Plan of Base Plate
1 Pl. 12"×$\frac{3}{4}$" 1'-4" lg.
1'-0"
27'-9$\frac{7}{8}$"
2Ls $2\frac{1}{2}$"×$2\frac{1}{2}$"×$\frac{1}{4}$" - 7'-7$\frac{3}{4}$" lg.
1 Pl. 12"×$\frac{5}{16}$" 0'-11$\frac{1}{2}$" lg.
1 L $2\frac{1}{2}$"×$2\frac{1}{2}$"×$\frac{1}{4}$" - 12'-7$\frac{1}{2}$" lg.
Ship Separate
2Ls $3\frac{1}{2}$"×$2\frac{1}{2}$"×$\frac{5}{16}$" - 15'-4$\frac{3}{4}$" lg.
2Ls $2\frac{1}{2}$"×$2\frac{1}{2}$"×$\frac{1}{4}$" - 7'-6$\frac{3}{8}$" lg.
1 Pl. 23"×$\frac{5}{16}$" 2'-1$\frac{1}{2}$" lg.
1 Pl. 13"×$\frac{5}{16}$" 1'-2$\frac{1}{2}$" lg.
1 Pl. 21"×$\frac{5}{16}$" 2'-6" lg.
Face Lower Edge
2Ls $3\frac{1}{2}$"×$2\frac{1}{2}$"×$\frac{5}{16}$" - 15'-9$\frac{1}{2}$" lg.
Ship Separate
2Ls 5"×$3\frac{1}{2}$"×$\frac{3}{8}$" - 16'-6$\frac{3}{4}$" lg.
2Ls 6"×4"×$\frac{7}{16}$"
$1\frac{3}{4}$" Hole Reamed
Rod Suspension
14'-0"
12'-0"
7'-11"
8'-0$\frac{1}{2}$"
8'-0$\frac{1}{2}$"
48'-0" C to C of Bearings

Fig. 267.

determined from the strain sheet and the value of the rivet; $\frac{3}{4}$-inch rivets are generally used, and gusset plates $\frac{5}{16}$- or $\frac{3}{8}$-inch. Where strains are very heavy and it is desired to avoid larger gussets, thicker plates can be used.

The measurements noted under (5) will be fixed by the above full-sized layout. It should be carefully borne in mind that such a layout is worse than useless unless it is very accurate, and therefore care should be taken to insure accuracy.

Special Notes and Details. As regards the shop marks noted under (7), each shop has a different practice. A convenient form, however, is to call the top chord "T. C. 1," "T. C. 2," etc.; the bottom chord "L. C. 1," "L. C. 2," etc.; the verticals "V 1," "V 2," etc.; the diagonals "D 1," "D 2," etc.

The exact size and the cuts of the gusset plates are generally left to the templet maker; they can be given, however, if it is desirable to do so, by adding the necessary measurements, which should be obtained from the full-sized layout of the joint.

Sometimes, in long trusses, it becomes necessary to draw the elevation of the truss as outlined above, and to supplement this by a larger-scale drawing of each joint, this larger drawing giving all the measurements of the connections as related to the panel point, and the smaller-scale elevation giving the general measurements.

Where it is not essential for appearance or for compactness of details to cut the angles on a bevel parallel to the abutting members, as is shown by some of the drawings, a square cut can be used and will somewhat simplify the shopwork.

Gussets should always be cut as closely as possible, both for neatness in appearance and for saving in weight.

In detailing, always show gussets, where possible, of such shape that they can be cut from a rectangular plate, using one or more of the sides of the original plate, and shearing off only where necessary for compactness of detail.

Compression members made of two angles should always be riveted together through a washer at intervals of two or three feet. In general, it is good practice to follow this for all members' tension as well as compression, as it stiffens the truss against strains in shipment and against possible loading not considered in calculations, and the extra cost is inconsiderable.

Illustrations of Shop Details. Fig. 268 shows a parallel chord truss carrying a floor, roof, and monitor load. Figs. 269, 270, and 271 show the connection of wood purlin under monitor girder to steel truss. The floor in this case rested directly on the top chord, which therefore brought bending strains as well as direct compression; for this reason the channel section was necessary. Note that for determining number of rivets in each member, one-half the stress would be considered, and the rivets taken at their single-shear value. Tie plates are used at intervals to stiffen the lower flanges of the channels forming the top chord.

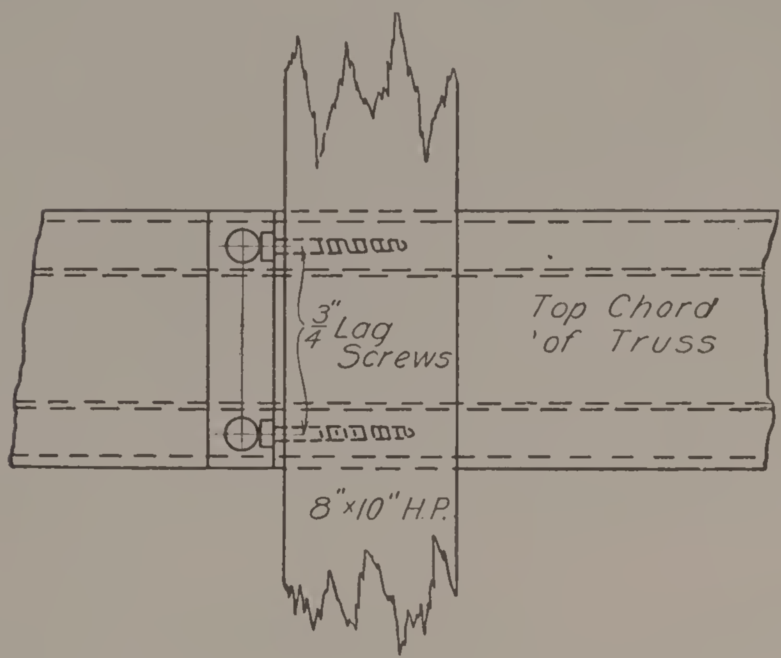


Fig. 269.

Fig. 272 shows the strain sheet for another parallel-chord truss 74 feet long, center to center of bearings. This truss carries a roof

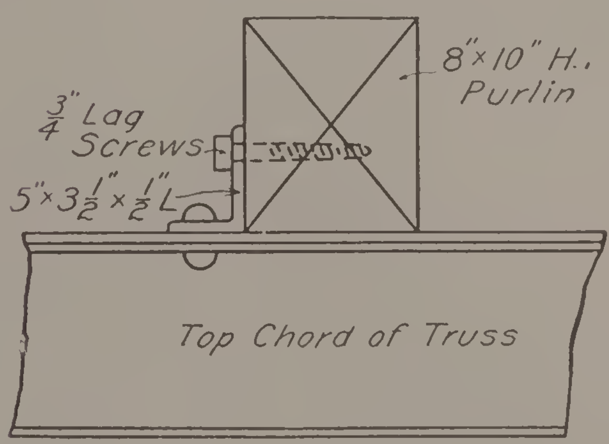


Fig. 270.

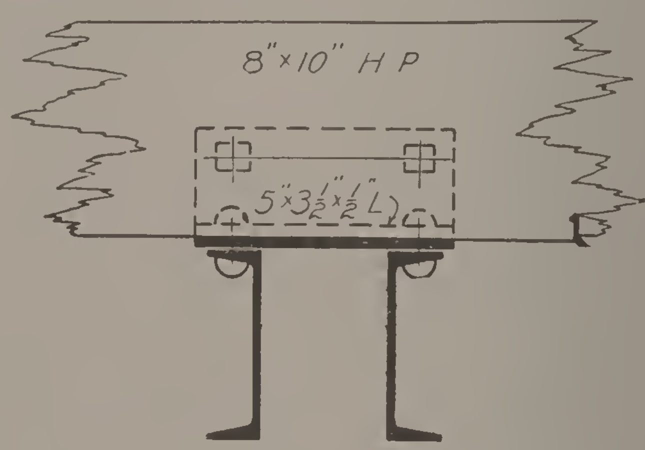
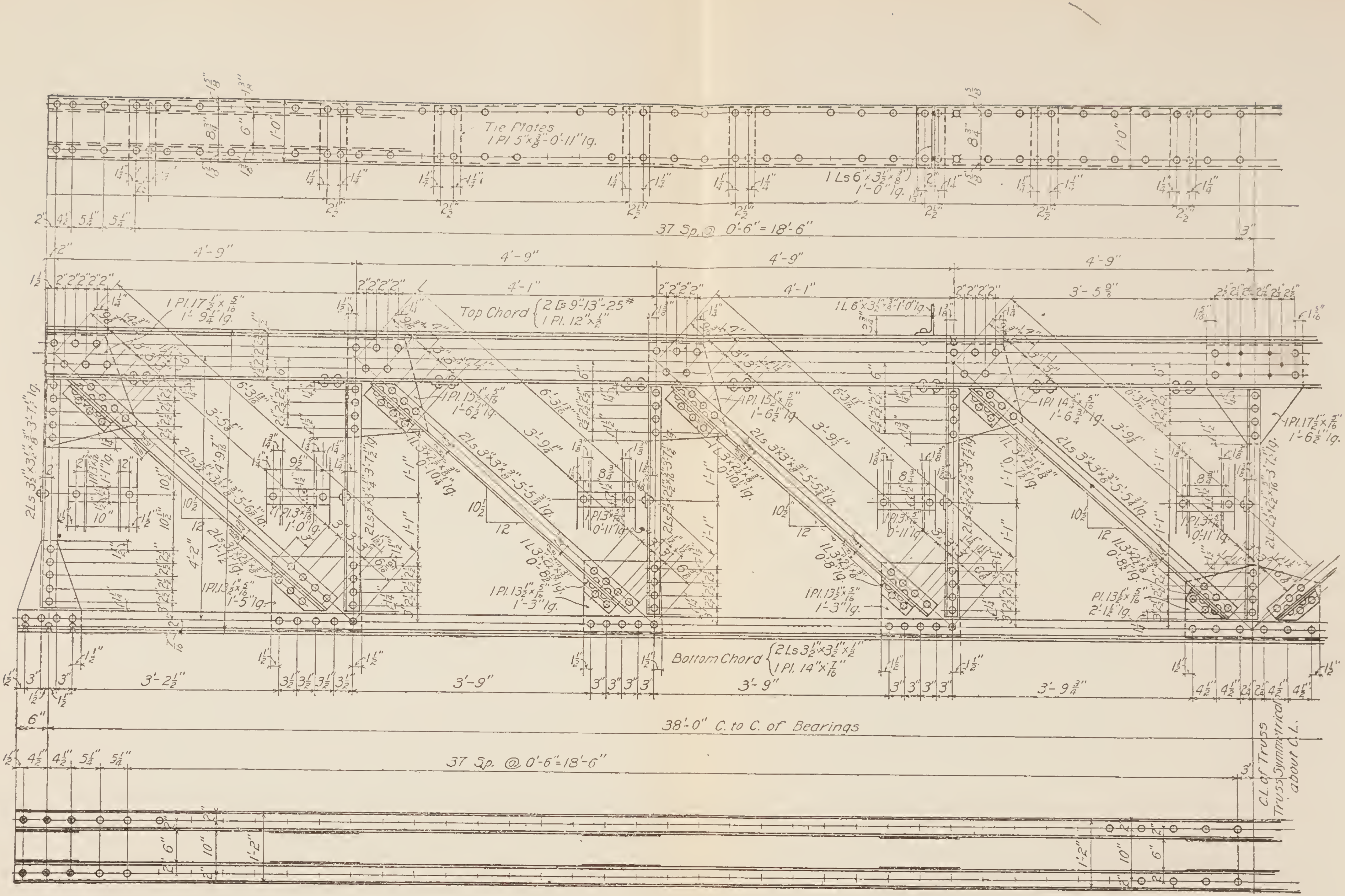


Fig. 271.

load assumed as 40 pounds live and 25 pounds dead per square foot, and also carries in the bottom chord a ceiling load of 15 pounds per square foot.

The roof beams span from truss to wall, which is 26 feet. On account of the construction and the long span, the wood framing is not considered as bracing the truss, which is therefore unsupported laterally except at the center where a steel strut is provided.



Tie Plates
1 Pl 5"×3/8"-0'-11" lg.
1 Ls 6"×3 1/2"×3/8"
1'-0" lg.
37 Sp. @ 0'-6" = 18'-6"
4'-9"
4'-1"
3'-5 9/16"
Top Chord {2 [s 9"-13"-25# / 1 Pl. 12"×1/2"
1 Pl. 17 1/2"×5/16" 1'-9 1/4" lg.
1 L 6"×3 1/2"×3/8"-1'-0" lg.
Bottom Chord {2 Ls 3 1/2"×3 1/2"×1/2" / 1 Pl. 14"×7/16"
3'-2 1/2"
3'-9"
3'-9 3/4"
38'-0" C. to C. of Bearings
C.L. of Truss
Truss Symmetrical about C.L.

Fig. 268

The manner of working out the stresses of such trusses by the analytical method, will be given below.

In all statically determined structures, there are three equations which must be true in order that the structure shall remain in equilibrium:

1. The algebraic sum of the moments, about any point, of all the external forces acting on the structure, must be zero. If this is not the case, there will be a rotation of the structure about this point.

2. The algebraic sum of all the external vertical forces must be zero.

3. The algebraic sum of all the external horizontal forces must be zero.

Both these latter conditions are evidently essential for the equilibrium of the structure.

In a truss loaded solely with vertical forces, the first two conditions are the only ones which would be used. If the truss is acted on by a wind load which has a vertical and horizontal component, then the third condition needs to be considered.

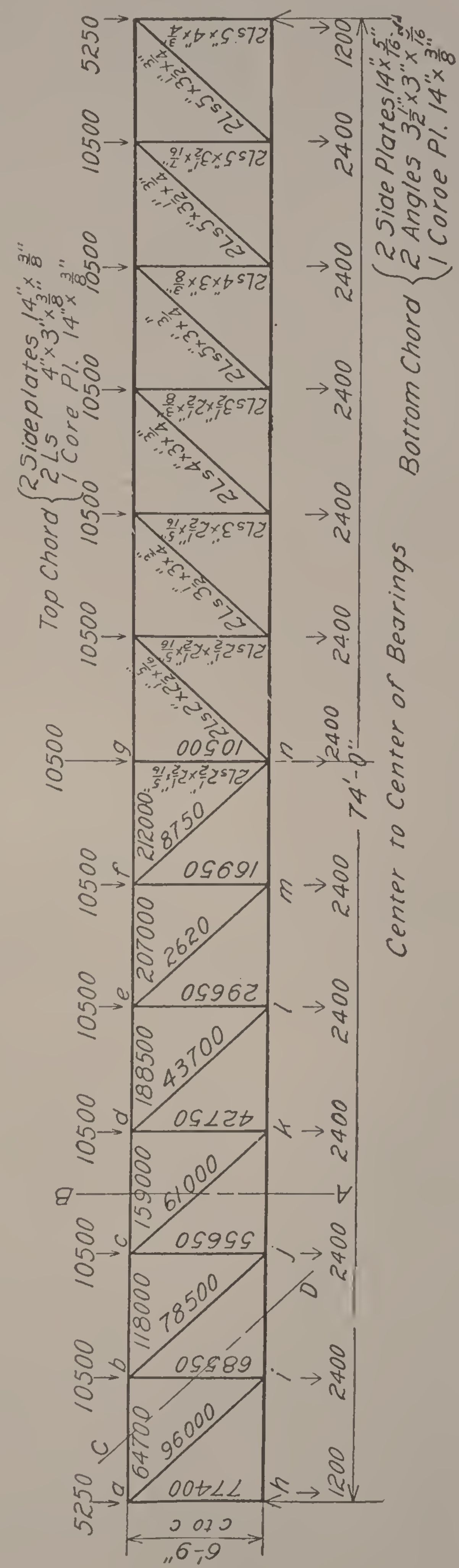


Fig. 272.

In the strain sheet given in Fig. 272, the first thing to determine is the panel load. The load at each top panel is 26.25 × 65 × 6.17 = 10,500; the bottom panel load is 26.25 × 15 × 6.17 = 2,400. Having determined these, and

noted them as indicated on the diagram. the only other external force to determine is the reaction. As the truss is symmetrically loaded, the reactions are equal, and each equal to half the total load, or 77,400 pounds.

Suppose the top and bottom chords and the diagonal of this truss were to be cut through on the line AB, as shown in Fig. 272. It is evident that, if the truss were then loaded as shown by the diagram, the portions of the top chord on each side of this cut would push against each other, and the portions of the bottom chord on either side would tend to pull apart, and the portions of the diagonal on either side would tend to pull apart. Unless there were some way of transferring from one side to the other these forces tending to push together and tear apart, the truss would not stand. It is therefore

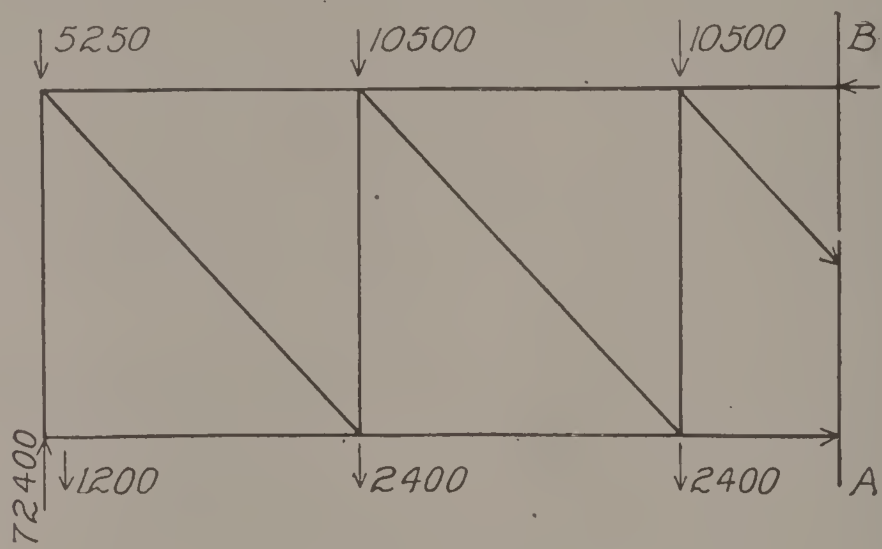


Fig. 273.

the reaction of the portion of the truss on one side of the section AB, acting upon the portion on the other side along the lines of the different members, which holds the truss in equilibrium. If therefore the portion of the truss to the right of AB is considered as taken away, and if, along the lines of the top and bottom chords and the diagonal, forces are applied of the same intensity as the forces which resulted from the reaction of the portion on the right and which held the truss in equilibrium, then these forces can for the time being be considered as external forces, and the intensity of them will be such as to fulfill the three conditions of equilibrium as regards the external forces. This condition is indicated in Fig. 273. It will be seen that these forces acting along the lines of the members of the truss cut by the section are the actual stress in these members necessary to maintain the truss in equilibrium. The stresses produced in the members of a structure

by the action of the loads, are called the "internal" or "inner" forces, in distinction from the "external" forces or "loads."

Any section, such as AB, cutting three members, gives three stresses to be determined. The top and bottom chord stresses are determined by using the condition that the algebraic sum of the moments about any point is zero. For the top chord, the point chosen is the intersection of the bottom chord and the diagonal. The moment of the stress in these two members about this point, is therefore zero, and this leaves only the moment of the top chord stress, which must then be equal to the moment of the loads about this point.

In a similar manner, taking moments about the intersection of the top chord and the diagonal, leaves only the moment of the bottom chord stress to be determined, which must equal the sum of the moments of the loads about this point.

In Fig. 272 these top and bottom chord stresses are determined by taking sections through the truss at the left of each panel point. These top chord stresses will be worked out below.

STRESS IN *ab*:

$$77{,}400 \times 6.17 = +\ 476{,}000$$
$$6{,}450 \times 6.17 = -\ \ 39{,}500$$

$+\ 436{,}500$ ft. lbs. = Moment to be balanced by moment of stress in top chord.

$$\text{Stress in } ab = \frac{436{,}500}{6.75} = +\ 64{,}700 \text{ lbs.}$$

STRESS IN *bc*:

$$77{,}400 \times 6.17 \times 2 = +\ 955{,}000$$
$$12{,}900 \times 6.17 \times 2 = -\ 159{,}000$$
$$\text{M of } bc = +\ 796{,}000$$

$$\text{Stress in } bc = \frac{796{,}000}{6.75} = +\ 118{,}000 \text{ lbs.}$$

STRESS IN *cd*:

$$77{,}400 \times 6.17 \times 3 = +\ 1{,}430{,}000$$
$$12{,}900 \times 6.17 \times 4.5 = -\ \ 357{,}000$$
$$\text{M of } cd = +\ 1{,}073{,}000$$

$$\text{Stress in } cd = \frac{1{,}073{,}000}{6.75} = +\ 159{,}000 \text{ lbs.}$$

STRESS IN *de*:

$$77{,}400 \times 6.17 \times 4 = +\ 1{,}910{,}000$$
$$12{,}900 \times 6.17 \times 8 = -\ \ 637{,}000$$
$$\text{M of } de = +\ 1{,}273{,}000$$

$$\text{Stress in } de = \frac{1{,}273{,}000}{6.75} = +\ 188{,}500 \text{ lbs.}$$

STRESS IN ef:

$$
\begin{aligned}
77{,}400 \times 6.17 \times 5 \quad &= +\ 2{,}390{,}000\\
12{,}900 \times 6.17 \times 12.5 &= -\ \underline{\ \ 995{,}000}\\
\text{M of } ef \qquad &= +\ 1{,}395{,}000
\end{aligned}
$$

$$\text{Stress in } ef = \frac{1{,}395{,}000}{6.75} = +\ 207{,}000 \text{ lbs.}$$

STRESS IN fg:

$$
\begin{aligned}
77{,}400 \times 6.17 \times 6 &= +\ 2{,}860{,}000\\
12{,}900 \times 6.17 \times 18 &= -\ \underline{1{,}430{,}000}\\
\text{M of } fg \qquad &= +\ 1{,}430{,}000
\end{aligned}
$$

$$\text{Stress in } fg = \frac{1{,}430{,}000}{6.75} = +\ 212{,}000 \text{ lbs.}$$

In explanation of the above, it will be noted that the moments of those forces causing right-handed rotation are designated "+" (plus), and those causing left-handed rotation are designated "−" (minus). Also note that the moment at any point consists of the moment of the reaction which for the left-hand reaction causes a positive moment and of the moment of the panel loads (including those over the end) which cause negative moment. As these panel loads are all equal, their moment can most easily be obtained by multiplying this panel load by the panel length and by the sum of the number of panels detween the origin of moments and the loads. Take for example the stress in cd; there is one full panel load distant one panel length, and a half-panel load distant two panel lengths; combined, these equal one full panel load distant two panel lengths.

As a check on the moment at the center, it is well to calculate in a different manner. As this is the point of maximum moment, this moment is the sum of the maximum moments which each load can produce. Or it is the sum of the reaction of each panel load, multiplied by the distance from the reaction to the panel point. Therefore, as a check, we have:

$$M = 12{,}900 \times 6.17 \times 18 = 1{,}430{,}000 \text{ foot-pounds.}$$

In a similar manner, the stresses in the bottom chord would be determined, taking moments about the top chord intersections with the diagonals.

There is a simpler way, however. If a section is taken along the line C D, and the portion to the right is removed as shown by Fig. 274, it will be seen that—just as was explained for the section A B—the forces acting along the lines of the members cut are the stress in these

members necessary to maintain equilibrium. Since the forces along *ab* and *ij* are horizontal, and are the only horizontal forces acting upon the structure, then these two must be equal in order to fulfill the condition stated—that the sum of the horizontal forces equals zero. This determines all the bottom chord stresses from the top chord stresses.

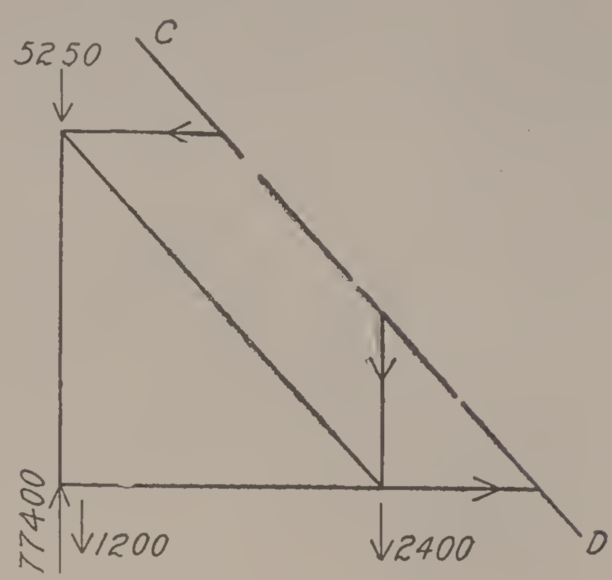


Fig. 274.

Direction of Stress. A stress acting toward the portion of the truss not considered removed, is *positive* and is *compression*. A stress acting toward the portion considered removed, is *negative* and is *tension*. The direction in which the stress must act is determined by the direction of the resulting moment of the external forces. If these produce right-hand rotation, then the stress in the member must produce left-hand rotation in order that the algebraic sum of the moments shall be zero. Therefore, in the case of the top chord stresses previously illustrated, since the resulting moment of the external forces is always positive, the moment of the stress in the chord must be negative or act toward the portion not removed, and the stress is therefore compression.

In the case of the bottom chord, this stress must act in the opposite direction to the stress in the top chord, and is therefore tension.

Stress in Verticals. This is determined by the condition that the algebraic sum of the vertical forces must be zero. Taking a section similar to C D, the only vertical force, aside from the loads acting on the truss, is the stress in the vertical member cut. This stress, therefore, equals the algebraic sum of the external forces on the left of this section, or the shear, and is opposite in direction or acts downward toward the portion of the truss not removed; the stress therefore is compression.

Stress in $ah = 77{,}400 - 1{,}200 = +\ 76{,}400$
" " $bi = 77{,}400 - 8{,}850 = +\ 68{,}500$
" " $cj = 77{,}400 - 21{,}750 = +\ 55{,}650$
" " $dk = 77{,}400 - 34{,}650 = +\ 42{,}750$
" " $cl = 77{,}400 - 47{,}550 = +\ 29{,}850$
" " $fm = 77{,}400 - 60{,}450 = +\ 16{,}950$
" " *gn* panel load $= +\ 10{,}500$

This latter stress in *gn* is obtained by taking the section around the panel point *g*, thus cutting only the top chord and the vertical. If the section was taken any other way through this vertical, it would cut a diagonal, and it would be necessary to determine the vertical component of this stress before the stress in the vertical would be known.

Stress in Diagonals. This is determined by taking sections similar to A B, and determining the vertical component of the stress in the diagonal. This vertical component must equal the algebraic sum of the vertical forces on the left, or the shear at the section. The relation of the actual stress in the diagonal to the vertical component, is the same as the relation between the length of the diagonal and the vertical depth. In this manner the stresses are worked out below:

Stress in $ai = 1.35 \times 70{,}950 = -96{,}000$
" " $bj = 1.35 \times 58{,}050 = -78{,}500$
" " $ck = 1.35 \times 45{,}150 = -61{,}000$
" " $dl = 1.35 \times 32{,}250 = -43{,}700$
" " $em = 1.35 \times 19{,}350 = -26{,}200$
" " $fn = 1.35 \times 6{,}450 = -8{,}750$

The direction of stress in these diagonals will be understood from Fig. 273, which shows the vertical component acting in an opposite direction to the resultant external forces.

Choosing the Sections. The fiber stresses used here are tension, 15,000 lbs.; compression, 12,000 lbs., reduced by Gordon's formula.

Both top and bottom chords are subjected to bending stresses due to the roof and ceiling joists, which come on these chords between the panel points. The bending stresses must be added to the direct stresses.

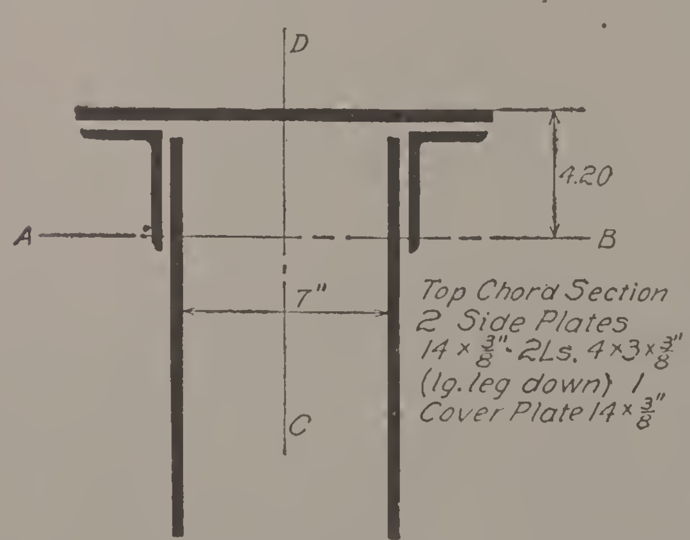


Fig. 275.

It is necessary at first to assume approximately what the direct fiber stress can be without exceeding the allowable stress reduced for unsupported length and for the bending stress. Having selected a section on the basis of this assumed fiber stress, the moment of inertia and the actual stress must be determined. If these vary materially from the allowable, a new

section must be chosen and the process repeated. In this case the process is illustrated below.

Top Chord. Fig. 275 shows the assumed section of top chord. The first step is to determine the position of neutral axis.

$$\begin{array}{lrl} \text{Cover plates} & 5.25 \times .19 = & 1.00 \\ \text{Side plates} & 10.5 \times 7.38 = & 77.50 \\ \text{Angles} & 4.96 \times 1.66 = & \underline{8.20} \\ & & 86.70 \end{array}$$

$86.70 \div 20.71 = 4.20 =$ Distance of neutral axis from top of cover plate.

Moment of Inertia of Top Chord.

$$\begin{array}{rl} Iab = 5.25 \times 4^2 & = 84.0 \\ \frac{1}{12} \times \frac{3}{4} \times \overline{14}^3 & = 171.0 \\ 10.5 \times \overline{3.18}^2 & = 106.0 \\ 3.96 \times 2 & = 8.0 \\ 4.96 \times \overline{2.54}^2 & = \underline{32.} \\ & 410.0 \end{array}$$

Radius of gyration $r = 4.4$

$$\begin{array}{rl} Icd = 5.25 \times \overline{3.69}^2 \times 2 & = 142.5 \\ \frac{1}{12} \times \frac{3}{8} \times \overline{14}^3 & = 85.5 \\ 1.92 \times 2 & = 3.8 \\ 2.48 \times \overline{4.66}^2 \times 2 & = \underline{107.8} \\ & 339.6 \end{array}$$

Radius of gyration $r = 4.05$

The top chord between panel points may be considered as a beam of span equal to panel length, and fixed at the ends as regards the bending moment caused by the direct load. Therefore,

$$\begin{aligned} M &= \tfrac{2}{3} \times \tfrac{1}{8} \times 65 \times 26 \times \overline{6.17}^2 \times 12 \\ &= 64{,}000 \text{ inch-pounds.} \end{aligned}$$

$$f_c = \frac{64{,}000 \times 4.2}{401} = 670$$

$$fsd = \frac{212{,}000}{20.7} = \frac{10{,}250}{10{,}920}$$

Since the top chord is braced laterally only at the ends and at three points equally distant, the unsupported length is 18 feet 6 inches. From Cambria, the allowable fiber stress in compression for a length of 18 feet 6 inches, and least radius of gyration 4.05, is found to be 11,000 lbs. reduced from 12,000 lbs. The above combined stress is therefore within the limit and close enough not to require redesign.

Bottom Chord. The bending moment is

$$\begin{aligned} M &= \tfrac{2}{3} \times \tfrac{1}{8} \times 15 \times 26 \times \overline{6.17}^2 \times 12 \\ &= 14{,}700 \text{ pounds.} \end{aligned}$$

Fig. 276 shows the assumed section of bottom chord. The neutral axis is determined as follows:

$$\begin{array}{rl} 2 \times 14 \quad \times \frac{5}{16} \quad \times 7.38 &= 64.6 \\ 2 \times \ 1.93 \times 1.44 \qquad &= \ \ 5.5 \\ 14 \times \ \frac{3}{8} \quad \times \ .19 \qquad &= \ \ 1.0 \\ & \overline{\ 71.1} \end{array}$$

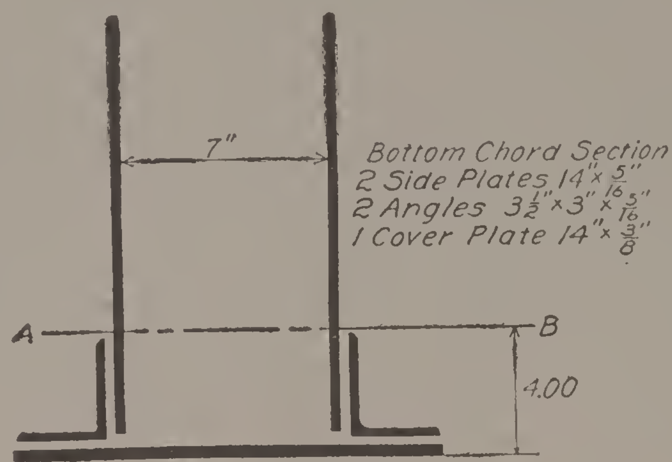


Fig. 276.

$71.1 \div 17.86 = 4.00 =$ Distance of center of gravity from bottom of plate.

Moment of Inertia of Bottom Chord.

$$\begin{array}{rl} Iab = \frac{1}{12} \times \frac{5}{8} \times \overline{14}^3 &= 105.0 \\ 8\ 75 \times \overline{3.38}^2 &= \ \ 99.6 \\ 2 \quad \times 2.33 &= \ \ \ 4.7 \\ 3.86 \times \overline{2.56}^2 &= \ \ 25.3 \\ 5.25 \times \overline{3.81}^2 &= \ \ \underline{76.1} \\ & \ \ \ 310.7 \end{array}$$

$$ft = \frac{14{,}700 \times 4.0}{310.7} = 189.$$

$$fsd = \frac{207{,}000}{14.11 \text{ (net)}} = \frac{14{,}650}{14{,}839}.$$

As the bottom chord is subject only to tension, it is not necessary to calculate the radius of gyration or moment of inertia about axis *c d*.

Diagonals are designed by using 15,000 lbs. tension, and choosing angles whose net section, taking one rivet hole out, will be sufficient for the stress in the member.

Verticals are designed by assuming an allowable fiber stress based on the reduction of 12,000 lbs. for ratio of length to radius of gyration. After the section is determined, using this assumed fiber stress, it is necessary to see that this fiber stress is within the actual allowable stress for the radius of gyration of the member.

Where two angles are used, spread the thickness of gusset plate, the least radius is employed, either parallel with the outstanding legs

or through the axis of the gusset. Where side plates are used, as in this case, the radius employed should be that parallel to the outstanding legs. These angles being spread and either laced or tied with plates, are weakest in the direction of the axis of the truss. The student should follow through the different sizes given for verticals and diagonals, fully understanding the above explanations.

Fig. 278 shows a detail of the connections at one top chord panel point; and Fig. 279, of one bottom chord panel point. It should be

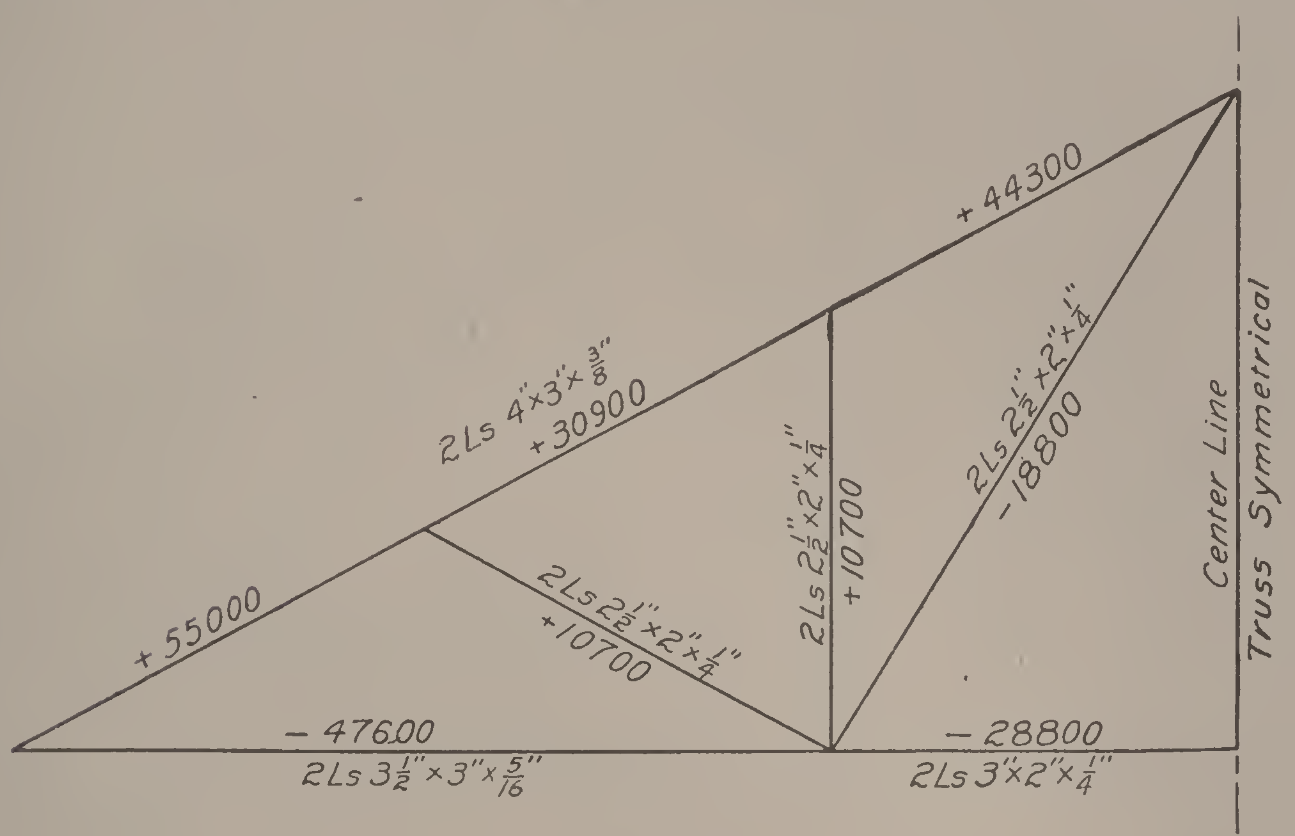


Fig. 277.

noted that the rivets are in single shear, and that the side plates are deep enough to allow connections to be made without the use of gussets.

In Fig. 267, a detail is shown of a connection suitable for a rod hanging, a balcony, or other member to the truss. Note that the center of rod comes at the intersection of the strain lines at the panel point. This should always be the case unless the chord is made specially strong to resist the bending due to a connection between panel points. Note also that the connection is applied directly to the gusset plate by a pin through the clevis nut. This brings only shearing and bearing strains on the connection, and avoids any direct pull on the heads of rivets or of bolts, which should be divided wherever possible in such cases.

The open holes in top chord are for securing the roof purlins to the truss. These purlins run directly across the top chord.

Fig. 280 shows the detail of a truss for a boiler-house roof. This roof has a high monitor running down the center, which is also framed with steel; the detail of this frame is shown in Fig. 284.

Fig. 284 shows a general view of the truss and monitor frame in position, and the roof beams framing to them. This truss was short enough to be riveted up at the shop and shipped whole. The monitor

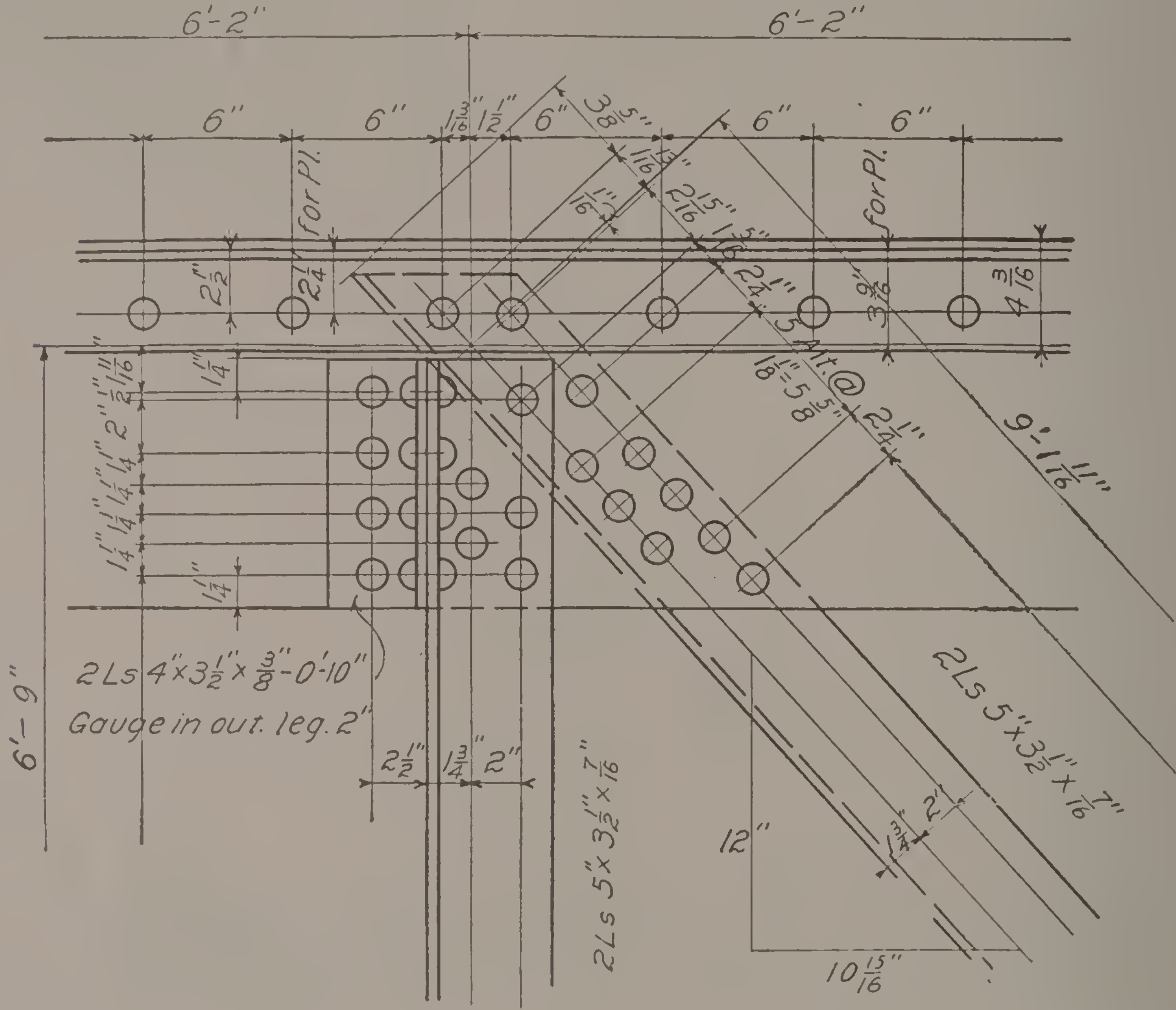


Fig. 278.

frame, however, was shipped separate from the truss, as indicated by the open holes for connection to truss. As this monitor frame, if shipped whole, would be likely to become bent and distorted, it had to be shipped in two parts, as indicated by the details.

Figs. 281 and 282 show the top and bottom chord splices in the center panel of the truss shown in Fig. 272. Note that the point in top chord is specified to be planed, and therefore the rivets provided are sufficient for only a portion of the stress, the balance being transferred by direct compression on the planed surfaces.

PROBLEMS

Determine all the stresses and suitable sizes to use for a truss loaded as shown in Fig. 283, and resting on a brick wall at each end. The load consists of floor joists resting directly on the top chord; and a 6 x 4 x $\frac{3}{8}$-inch angle should be provided near every other panel point, punched for lag screws to secure to wood joists for forming a lateral support to truss.

Make a complete shop detail of the above truss.

Trussed Stringers. Figs. 285 and 286 show the two common forms of trussed wooden stringers. These consist of a wooden beam,

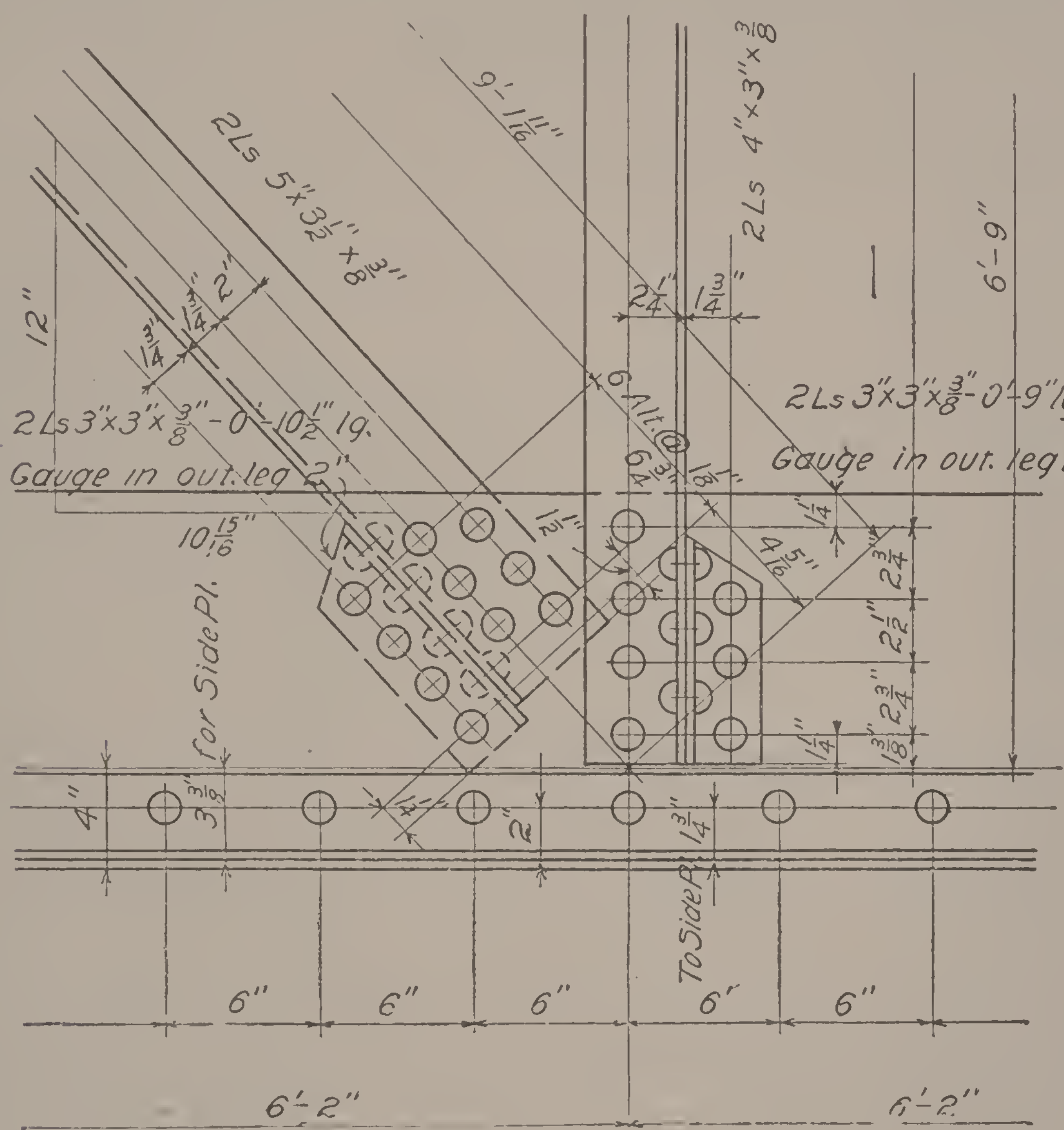


Fig. 279.

composed of one or more timbers, stiffened by one or two struts bearing on steel rods, as shown. They are used in timber-framed structures where it is impracticable to obtain timbers sufficiently strong to support the loads.

The trussed stringer is not a true truss, and the stresses cannot be accurately determined by the methods used for trusses, because the

stresses in the members depend upon the deflection of the beam as a member of a truss and as a beam also. The exact solution is very

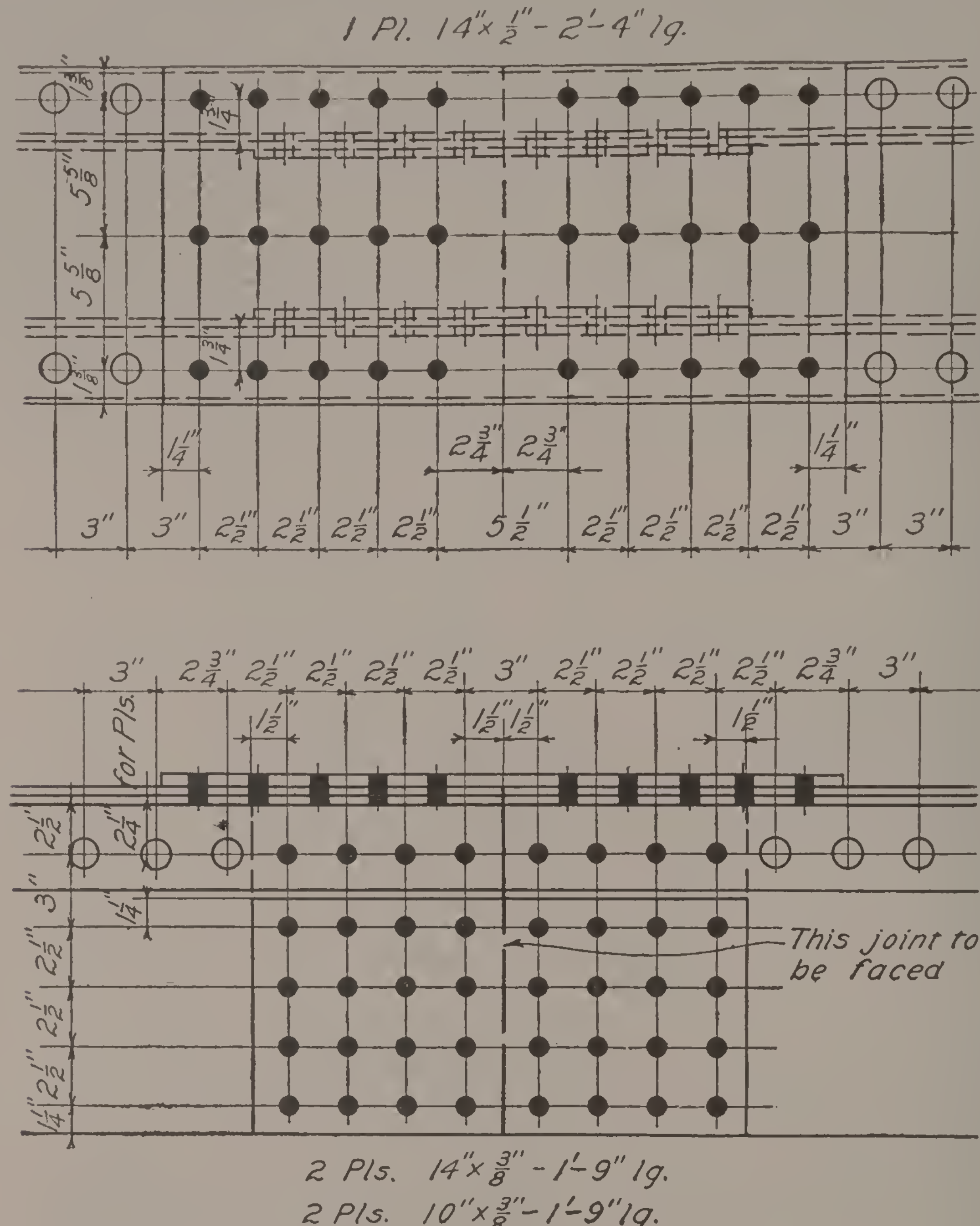


Fig. 281.

complicated. An approximate solution can be made as follows:

In Fig. 285, if a load P is applied over the center strut as shown, then

$$\text{Stress in } ac = \frac{P}{2} \times \frac{ac}{dc};$$

$$\text{Stress in } ab = \frac{P}{2} \times \frac{ad}{ac}; \text{ and}$$

$$\text{Stress in } dc = P.$$

If the load P is applied uniformly over the whole length of *ab*, then the stresses are approximately as follows:

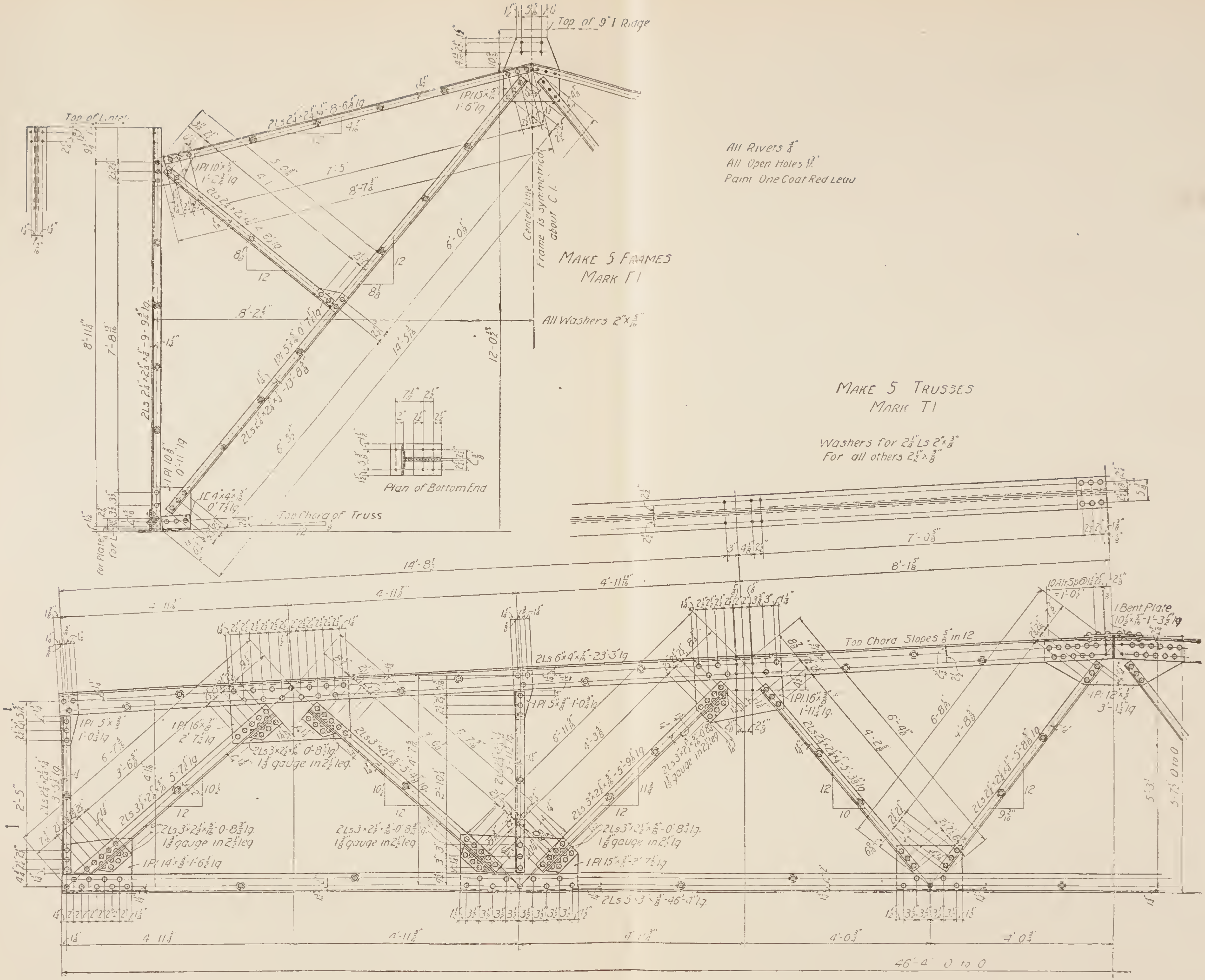

Top of 9" I Ridge
Top of Lintel
All Rivets 3/4"
All Open Holes 13/16"
Paint One Coat Red Lead
Center Line
Frame is symmetrical about C.L.
MAKE 5 FRAMES
MARK F1
All Washers 2"x1/2"
Plan of Bottom End
Top Chord of Truss
MAKE 5 TRUSSES
MARK T1
Washers for 2 1/4" Ls 2"x3/8"
For all others 2 1/2"x3/8"
1 Bent Plate
Top Chord Slopes 5/8" in 12
2Ls 6"x4"x7/16"-23'-3"lg
2Ls 5"x3"x3/8"-46'-4"lg
46'-4" 0 to 0

Fig. 280.

Load at d $= \frac{5}{8}$ P;

Stress in ac $= \frac{5}{16}\ \mathrm{P} \times \frac{ac}{dc}$;

Direct stress in $ab = \frac{5}{16}\ \mathrm{P} \times \frac{ad}{ac}$; and

Stress in dc $= \frac{5}{8}$ P.

The beams ad and db are however subjected to bending stress due to the load acting directly on the beam between the unsupported points

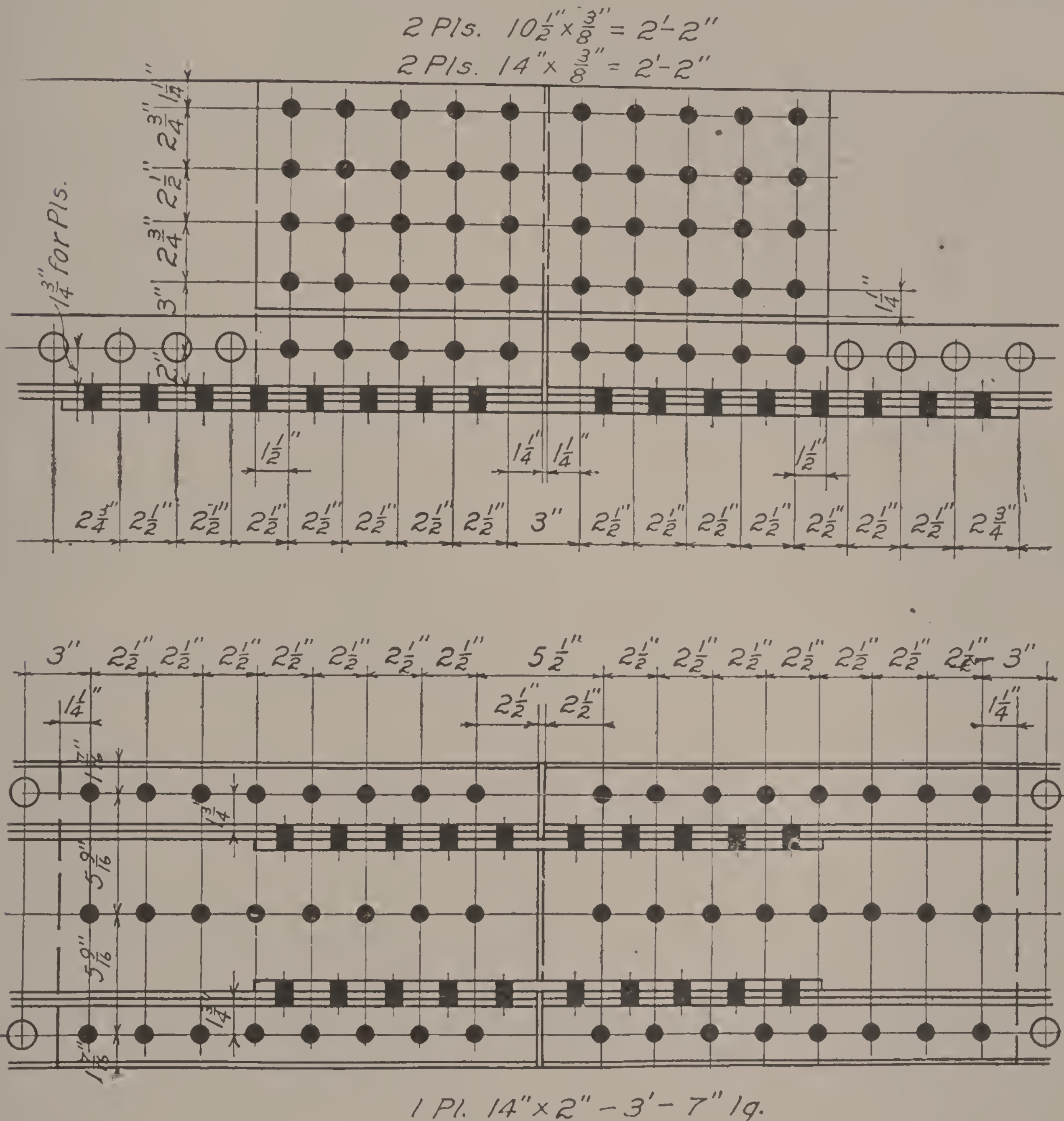


Fig. 282.

a and d and b and d. If I is the moment of inertia of the beam, this bending stress can be found approximately from the formula $f = \frac{\mathrm{M}y}{\mathrm{I}}$, in which y = Half the depth of the beam.

The bending moment may be taken as $\frac{3}{8}$ P $\times$ *ab*.

The beam must be proportioned so as to provide for the direct

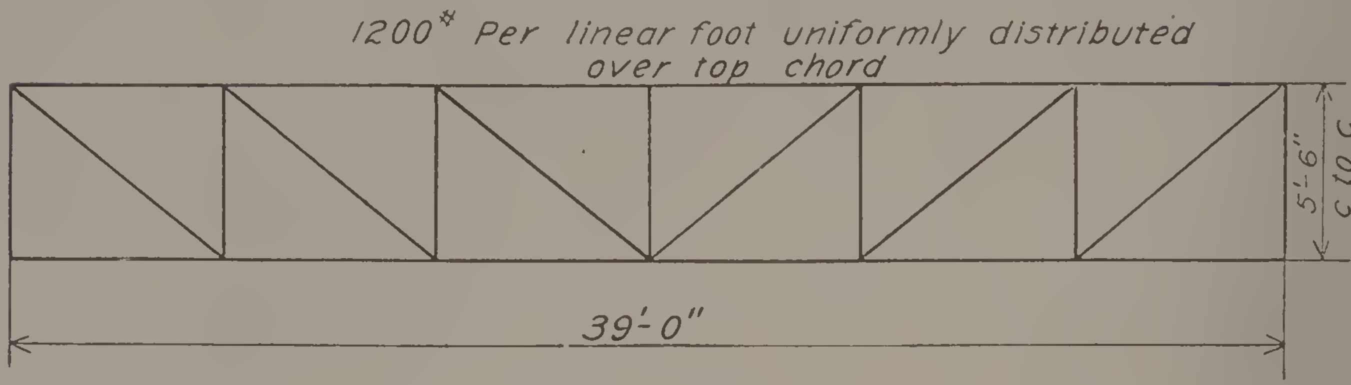


Fig. 283.

stress *plus* the stress due to bending, without exceeding the allowable fiber stress of the timber.

In Fig. 286, if a load P is applied over each of the struts, the stresses can be determined approximately as follows:

Stress in $ac = \mathrm{P} \times \frac{ac}{ec}$;

Stress in $ae = \mathrm{P} \times \frac{ae}{ac}$; and

Stress in $ec = \mathrm{P}$.

If the load 2 P is applied uniformly over the whole length *ab*, then the stresses are approximately as follows:

The load at *e* and *f* can be taken approximately as $\frac{5}{6}$ P; then

Stress in $ac \quad = \frac{5}{6}\,\mathrm{P} \times \frac{ac}{ec}$;

Direct stress in $ae = \frac{5}{6}\,\mathrm{P} \times \frac{ae}{ac}$; and

Stress in $ec \quad = \frac{5}{6}\,\mathrm{P}$

The portions *ac*, *cf*, and *fb* are subjected to bending stresses as

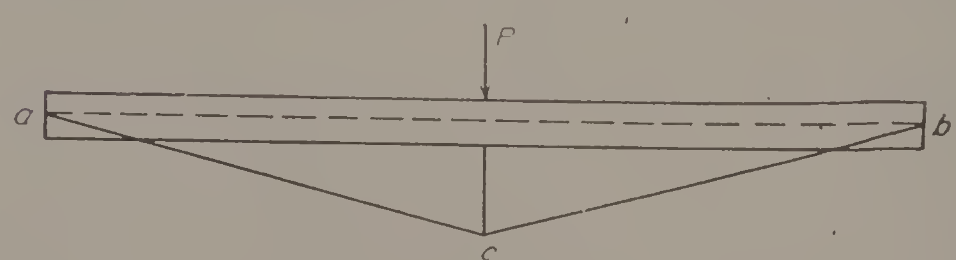
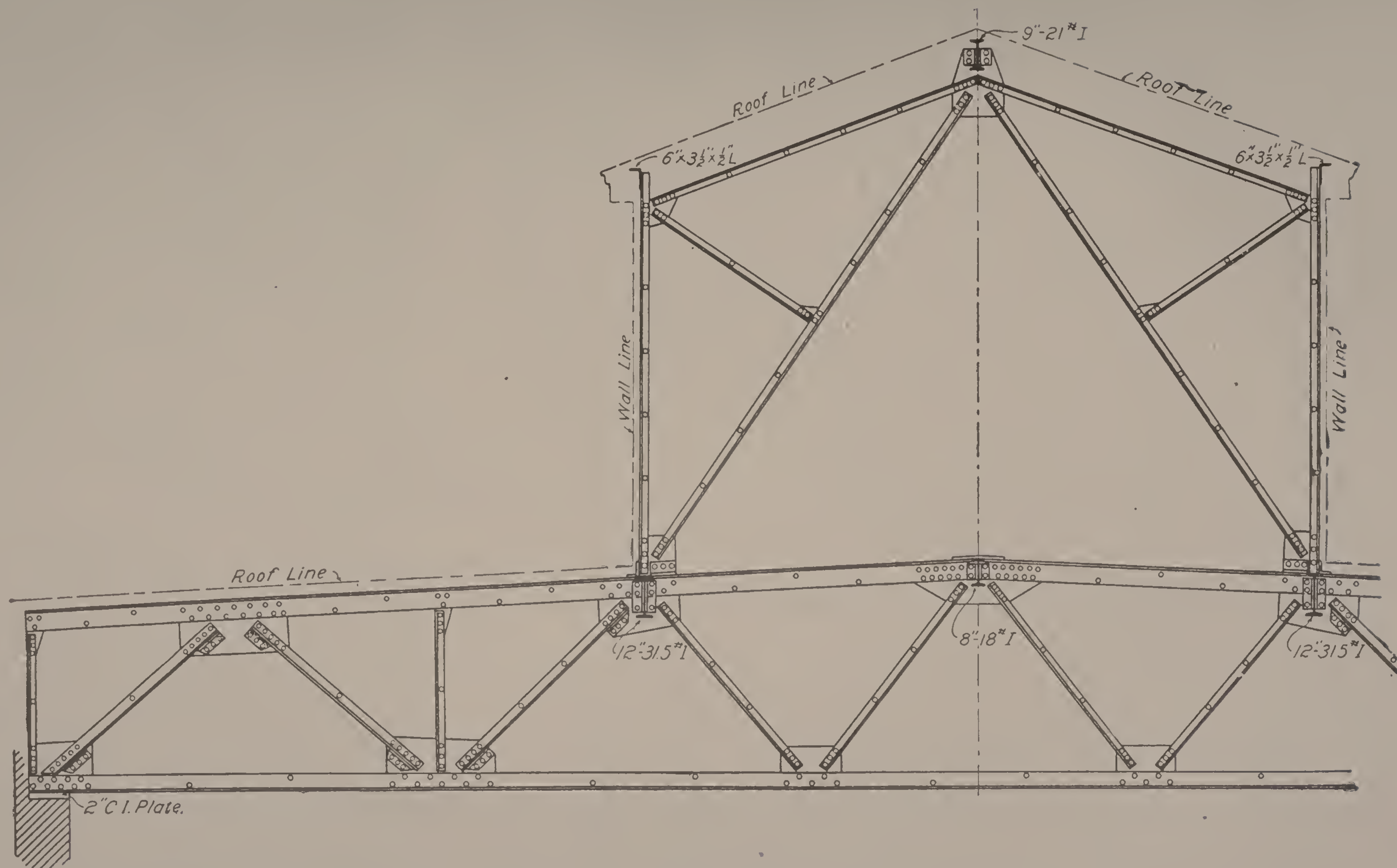


Fig. 285.

before; and if I is the moment of inertia of the beam, the bending stress in $ac = \frac{\mathrm{M}y}{\mathrm{I}}$, in which $y = \frac{1}{2}$ Depth of the beam; the bending M may be taken as $\frac{1}{3}$ P $\times$ *ab*. The beam must be proportioned so that



9"-21# I
Roof Line
Roof Line
6"×3½"×½" L
6"×3½"×½" L
Wall Line
Wall Line
Roof Line
12"-31.5# I
8"-18# I
12"-31.5# I
2" C.I. Plate.

Fig. 284.

the combined bending and direct stress shall not exceed the safe fiber stress for the timber.

Owing to the fact that the actual distribution of stress in trussed stringers is uncertain, and the methods of determining these stresses only approximate, a factor of safety of not less than 5 should be used.

The detail of the connection of the rods with the end of the beam

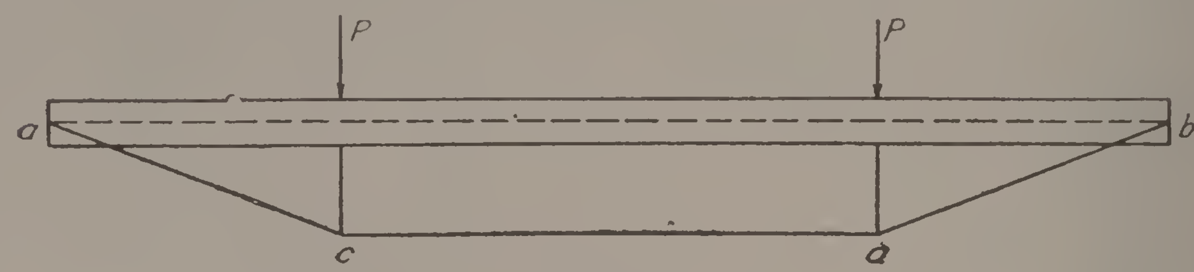


Fig. 286.

is shown in Fig. 287. Sometimes a single rod going between a horizontal beam made of two timbers, is used; and sometimes where two rods are used, these are placed outside of the timber. A detail which will avoid boring through the timber is preferable. The plate at the end must be large enough to distribute the stress without exceeding the

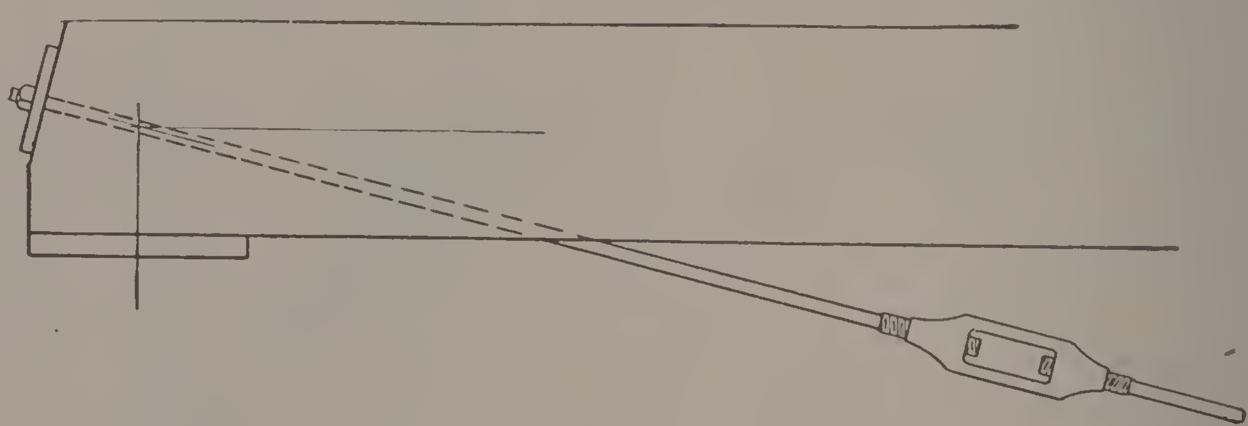


Fig. 287.

safe compression value of the timber used; for hard pine, this should be 1,000 pounds per square inch. The plate should be thick enough to provide for the shearing stress on the metal, and the bending stress induced by the pull of the rod on the unsupported portion of the plate.

It is important to have the center lines of the members intersect at the center of the bearing, as otherwise considerable additional bending stress will be caused, owing to the eccentricity.

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